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Innovative Means of Dealing with Potential Sources of Ground Water Contamination Proceedings of the National Ground Water Quality Symposium (7th) Held at Las Vegas, Nevada on September 26-28, 1984

National Water Well Association, Worthington, OH

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Robert S. Kerr Environmental Research Lab.
Ada, OK

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Innovative Means of Dealing With Potential Sources of Ground Water Contamination

Proceedings of the Seventh National Ground Water Quality Symposium

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The Seventh National Ground Water Quality Symposium was held in Las Vegas, Nevada, on September 26-28, 1984. The symposium was dedicated to the memory of Mahdi S. Hantush (1921-1984), a pioneering scientist who specialized in the application of mathematics to solve transient ground water flow problems. The theme of this conference was "developing and implementing innovative means of dealing with potential sources of ground water contamination." Among the potential sources of contamination addressed at the meeting were: underground storage tanks; mining and mine wastes; agricultural chemicals and land application practices; injection wells; abandoned wells; and hazardous and non-hazardous wastes in landfills and impoundments. Government officials, consulting engineers, geologists and hydrologists, planners, industry representatives, academicians and others met to discuss these and other topics and to share their experiences in these areas. These proceedings are a compilation of the papers presented by the symposium speakers.

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Special thanks are due the staff members of NWWA whose efforts provided for a smooth-running conference and without whom these proceedings would not have been possible.

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Abstract

The Seventh National Ground Water Quality Symposium was held in Las Vegas, Nevada, on September 26-28, 1984. The symposium was dedicated to the memory of Mahdi S. Hantush (1921 - 1984), a pioneering scientist who specialized in the application of mathematics to solve transient ground water flow problems. The theme of this conference was "developing and implementing innovative means of dealing with potential sources of ground water contamination." Among the potential sources of contamination addressed at the meeting were: underground storage tanks; mining and mine wastes; agricultural chemicals and land application practices; injection wells; abandoned wells; and hazardous and non-hazardous wastes in landfills and impoundments. Government officials, consulting engineers, geologists and hydrologists, planners, industry representatives, academicians and others met to discuss these and other topics and to share their experiences in these areas.

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Dedication

Mahdi Salih Hantush
1921 - 1984

Moid Ahmad

Ohio University
Athens, Ohio

Mehdi Salih Hantush, distinguished hydrologist and scientist, died January 14, 1984. He was born in Iraq and graduated from the American University of Beirut, Lebanon in 1942 in civil engineering. In 1947 he earned an MSC in Civil Engineering (irrigation engineering) from the University of California, Berkeley. He obtained his Ph.D (1949) in ground water hydrology from the University of Utah under Professor C. E. Jacob and Dean Peterson.

Between 1950 and 1958, he worked as a professor of irrigation engineering and the dean of the College of Engineering, University of Baghdad, Iraq and was a visiting professor at the Department of Geophysics, University of Utah, Salt Lake City and at the Department of Hydrology, New Mexico Institute of Mining & Technology, Socorro, New Mexico. In this period, he was actively involved in hydrological projects in Iraq and quantitative evaluation of the Roswell Reservoir, New Mexico.

Between 1958-1966, he was a professor of hydrology and the head of the ground water hydrology department at New Mexico Institute of Mining and Technology. He started the first ground water hydrology graduate program in U.S.A. He attracted first rate students from the United States and abroad. He published numerous original papers in the Journal of Geophysical Research, Water Resources Research, Trans American Geophysical Union, ASCE Transactions, and other journals. His classic work, "Hydraulics of wells", was published in Advances in Hydroscience by V. T. Chow. His analytical models include the theory of leaky aquifers, aquifer tests on partially penetrating wells, interference of wells, salt water intrusion and more. His research led to the development of the modern theory of leaky aquifers. Robert W. Stallman described Hantush as a "master of radial flow". We find very few papers on the subject that do not include his work. In this day of numerical and computer modeling, his equations are used as bench marks to caliberate the models. In 1968, he was awarded the O. E. Meinzer award of the hydrology division of Geological Society of America for distinguished work in the field of hydrology.

In 1968 he returned to Iraq and did consulting work for ground water development in Kuwait. He designed the Shigayya Brackish Water Field so that it could pump 60 million gallons per day for domestic use. He also developed mathematical models for the Kuwait ground water reservoir. These models are in the files of the ground water department, and time has shown that the well field is behaving according to his prediction. He was not only a superb theoretician but a great practitioner of hydrology. His concept of average transmissivity is very useful, for one can look to one well as representative for an entire well field. We have found that these concepts work in other well fields.

Between 1971-1978, he was a professor of applied math at the University of Kuwait. From 1978 until his death he was a professor of hydrology in the Department of Civil Engineering there. He taught water resources, hydrology and applied math. During this period he completed theoretical analyses of 40 research problems. Unfortunately these papers are unpublished, although he intended to publish them. We have lost a great hydrologist but his research will remain with us forever.

He suffered heart pains after an operation and doctors found blocked arteries. He is survived by five children and his wife, Iqbal Hantush.

I would now repeat the following verses written by Dr. Said U. Abusada who was a former student of Professor Hantush.

Consciously or unconsciously
you moved from the river
to the muddy bank
where there was not much life or growth

You clutched your roots
in the salty soil
and by your sweet sweat
you mashed the soil

Consciously or unconsciously
you put your steps
on the road of eternity
and you sharpened with your soul
and the power of your mind
a mass of analysis
to which the sword is no peer

You marched on a road
that was leveled by men of honour
for the nation's buds
to take off to eternity

To the heart of the modern west
you moved -
to a desert very similar
to that of your birthplace,
to a place where those who mash the salty soil
by their sweet sweat
are abundant
to inhale more knowledge
and to coin from your blood
the price of immortality

Here and there,
and everywhere,
the master of flow,
crowned you were

Away from your roots
you could not stay
for your buds had to grow
therefore, you had to return

In the small, but big-hearted country,
Kuwait,
you loved the sea,
the pure medium,
and there you gave knowledge,
the fruits of your life journey

Neither a green horn
nor an unthankful person
could emitter your heart
or curtail your contribution

On the white bed
the smile and the sense of humor
were always there
filling the place

At the graveyard
crowds of professionals gathered
to bid you their last farewell.
The querry still hovers
in the frustrated mind:
as today brings all of you together,
on an occasion of death,
tomorrow will do the same
for the nation's resurrection.

Keynote Speakers

Harold E. Thomas

I realize that my historical perspective must be a prehistoric perspective to most of you. I have spent more than 40 years trying to quantify the storage, inflow and outflow of ground water in reservoirs, their variations in time, and their part in the hydrologic cycle. I used to talk of ovedraft or ground water mining vs. safe yield, cyclic storage, and conjunctive use of reservoirs, replenishable resources, which I see now as part of the world of deficits and balanced budgets. Half a dozen years ago I wrote an article for Ground Water entitled "Cyclic Storage, Where Are You Now"? No answer. As to dissolved constituents I have been fortunate to work and live among safe drinking water--not pure, but natural and untainted by man or beast, so far as I know.

As I warned Dr. Lehr, I've not kept up with hydrologic literature since Watergate. All I know is what I read in the papers, as Will Rogers used to say. And all my reading has been done with old eyes. I offer comments on two major topics: Point sources in the urban San Francisco Bay Area, and nonpoint sources in the agricultural San Joaquin Valley--both in California.

San Francisco Bay Area

Santa Clara Valley extends southeastward from San Francisco Bay for about 50 miles. Fifty years ago the valley had thousands of acres of orchards, chiefly prunes and apricots; its largest city was San Jose (50,000) and industries were chiefly agricultural. Today the valley has perhaps 1 1/2 million people of which a million are in San Jose and the urban area extending northwest along the bay shore past Stanford University--the queen bee of high tech: This is Silicon Valley.

I remember the old agricultural valley, with 2,000 wells irrigating 200,000 acres, with overdraft offset successfully by artificial recharge into gravel beds along tributary creeks from the adjacent mountains. Now many of the old wells are still in use, but many others have been abandoned because the SCV WCD now imports about 100,000 acre-feet of water through the South Bay aqueduct of the State Water project, and many cities to the north purchase water from San Francisco.

Used water is discharged chiefly through sewage-treatment or waste-water treatment plants and then to the bay, although some raw sewage is carried in flood waters from the large cities. The hightech toxic wastes include trichlorethylene (TCE), dichlorethylene (DCE), trichlorethane (TCA), polychlorinated biphenols (PCBS), boards, chips and electroplating. These are unacceptable for sewage-treatment plants and many companies therefore use plastic tanks on their property for temporary storage of toxics and ultimate disposition by trucks.

In 1979 IBM reported a leak in a storage tank on its property in south San Jose, and in 1981 Fairchild reported a leak also in San Jose but more newsworthy because TCE had reached ground water and might contaminate the wells supplying 1,600 homes. Fairchild has spent more than $2 million and anticipates a total of $15 million may be required to recover and dispose of the spilled toxics. In 1982 California's Bay Area Regional Water Quality Control Board wrote to 2,600 companies and received replies from 1,400 who had storage tanks; the Board required about 100 of those who had toxic waste to test their tanks for leakage, and received word that at least 75 were suspect--a number that has subsequently been increased to 120. Most of the other tanks in the group of 1,400 acknowledging ownership of underground storage tanks are at gasoline service stations or other small businesses.

At the beginning of 1984 the California Dept. of Health Services published a list of the States' worst toxic chemical sites. Eleven of these are in Silicon Valley, but only five have hightech toxic wastes; Fairchild (no. 17 on the list) IBM (no. 21), Westinghouse in Sunnyvale (no. 28), Watkins-Johnson in Palo Alto (not listed) and Teledyne in Mountain View (no. 85). The others are older and several are along the former shore of the bay: one had a good headline--"Arsenic and Old Waste" in EPA (here referring to East Palo Alto). And several are in places where the economic value of the cleanup may be questioned.

Beginning in 1984 California law requires that for underground storage of toxic wastes, a storage tank must be of fiber glass and double-walled so that the interspace can be monitored for leakage from the inner tank. Unfortunately this law does not apply to existing tanks which are causing problems today, because of lobbying by oil interests. Very soon after the law was passed, a gas station tank in Belmont was found to have leaked 14,000 gallons of gasoline, and some of it was floating upon the water in three nearby domestic wells. The fire authorities ordered removal of the rusted steel tank and replacement by FIBERGLAS and the State Board of Health has required the owners to report on the extent of the contamination.

From what I read in the papers, the contamination of ground water by toxic wastes in Silicon Valley is occuring, and the potential for more leakage is known from surveys by mail. But it is in early stages, and the law requiring "leakproof" storage tanks can be a vital step in controlling leakage, particularly if owners of inadequate tanks are held responsible and negligent when leakage occurs.

From newspaper accounts the chief expertise in assessing the underground extent, distribution, depth of toxic wastes in soil and ground water has been by private industry which has brought in geohydrologic consultants. The understaffed State agencies have assessed the magnitude, identified point sources and monitored wells for public supply and some private wells when contamination has been reported or suspected. The region does appear to be an aggregate of point sources which could benefit by geohydrologic integration and coordination of the bits and pieces.

Federal Authority

The Lawrence Livermore National Laboratory is in the eastern part of livermore Valley, alluvium-filled and about 20 miles east of San Francisco Bay. Some of its wells, used only for monitoring ground-water quality, were found to contain PCE (perchlorethylene) and TCE in April 1983. Eight months later private wells close to the west boundary of the lab grounds were found to be tainted, and the landlord ordered his tenants off immediately, fearful that he would be held responsible for any damages from the well waters. Meantime the lab is providing free bottled water to them. Public-supply wells about 2 miles farther west have been tested and found to be clean so far. There is some difference of opinion between the hazards chief of the lab and the engineer of the Regional Water Resources Control Board: a northwesterly direction of flow is preferred by the lab: The tainted wells are to the southwest of the lab grounds. The Federal government has just announced its plans to condemn and take title to 75 acres formerly owned by three landowners, as a first step in creating a "buffer zone" for security.

Mather Air Force Base, ten miles east of Sacramento, is known to have disposed of hazardous chemicals at various sites, including a large drainage ditch, from the 1940's to the 1970's, near the western edge of the base. Some residents at the wells yielding tainted water have been warned not to drink it. But the Air Force is not yet ready to offer free bottled water: "The problem is off-base, but we're not sure that it's us." Additional monitoring wastes are needed, and the State has budgeted $60,000 for drilling outside the base, but the air base must demand funds from the Air Tactical Command.

San Joaquin Valley

The story of ground water in San Joaquin Valley can be told only by including all phases of the hydrologic cycle in the discussion, and some of the data I've found are quoted for the entire Central Valley.

Take ethylene dibromide (EDB): Last January Florida exported to California fruit which reportedly contained 200 times Florida's own limit for EDB. California promptly quarantined any EDB-treated fruit for twenty-one days and then in August halted the use of EDB in Merced, Stanislaus, and Kern Counties in California after the chemical was found in domestic and irrigation wells. In the same month Mendicino County banned spraying of phenoherbicides, which contain dioxin, and was sustained by the State Supreme Court. The State Legislature voted to overrule, and implied that no citizen shall be denied the right to spray or be sprayed according to State Policy. So we are sprayed for medfly, weeds and marijuana; and the Vietnamese who have moved to California find it very much like home. As another example the pesticide dibromochbropropane (DBCP) was banned nationally in 1979 as a suspected carcinogen. That year 40 wells were found to be contaminated by the chemical. In the Central Valley 400 wells were found to be contaminated by DBCP in 1982 and over 2,000 wells in 1983.

San Joaquin Valley is the middle third of California's Central Valley, a structural valley about 400 miles long and 50 miles wide, bordered on both sides and ends by mountains. The San Joaquin River rises in the highest part of the Sierra Nevada, enters the Valley just north of the city of Fresno and turns northwest to join the Sacramento in the Delta and flow out of the Central Valley into San Francisco Bay. The Kings River and Kern River rise in the Sierra farther south and flow into a closed basin which has not overflowed into the San Joaquin River for more than a century. The Tulare Lake closed basin receives the least precipitation (less than 10 inches annually) and least runoff from the mountains. The San Joaquin Valley receives more precipitation and sufficient runoff from Sierra streams that the river has a surplus as it joins the Sacramento in the Delta and outflow to San Francisco Bay. The Sacramento Valley is the part of the Central Valley with the greatest rainfall and greatest inflow from the surrounding mountains. California's water imbalance--too much in the north for the reasonable use of all northerners, and too little in the south for the needs and wants of the growing megalopoles of Los Angeles--has bothered politicians for half a century. The first corrective measures started half a century ago as the Central Valley Project: undertaken by the U.S. Bureau of Reclamation, it includes the Shasta Dam on the Sacramento River plus many others on streams rising in the Sierra Nevada; the Delta-Mendota canal which carries water lifted from the Delta to and beyond Mendota in San Joaquin Valley: The Friant-Kern Canal from San Joaquin river south into the Tulare Lake closed basin; and distribution systems to a hundred water contractors. Subsequently the California State Water Project (SWP) was undertaken to provide water for areas farther south than the area of CVP: lands in the Tulare Lake closed basin, and urban areas in Southern California.

The SWP includes the Oroville dam and reservoir on the Feather River and the California Canal west of and higher than CVP's Delta-Mendota canal and extending to the Edmonston pumping plant where water is lifted over the divide to coastal California. Deliveries began in 1975.

My oldest clipping (Nov. 1980) reports a meeting of representatives of several Water Districts along the upper part of the Delta-Mendota canal: The Panoche Water District has required each farmer to put in a siltation basin to reduce the sediment in drains; Broadview District requests permission of Grasslands to deepen 11 miles of drainage channels across Grasslands District to lower Broadview's water table. All agreements between men who have knowledge and can count on the quality of their water.

Another clipping from the Gustine Standard in November 1980 expressed consternation about the San Luis Drain which is 85 miles long, and terminates near Gustine. Constructed by the U.S. Bureau of Reclamation, its purpose was to collect drain water from lands irrigated by CVP. The clipping is dismayed that this drain picks up the agricultural chemicals--pesticides, herbicides, fertilizers--plus any salt residues dissolved during irrigation--and dumps them in Kesterson Reservoir. The drain was planned to continue on to San Francisco Bay, but this roused strong opposition in the Bay Area. Instead the Bureau of Reclamation constructed thirty-two 100-acre evaporation ponds at Kesterson Reservoir to reduce the water. But: "What about the possibility of the ponds filtering through the "seal" and ruining the huge aquifer of pure ground water that Kesterson lies over?" So far, no one is trying to find an answer to this question.

In 1983 flood waters collected in the Tulare closed basin and gave Tulare Lake an area of 80,000 acres--acres which had been dry and farmed by irrigation. Two agribusiness corporations who own about 60,000 acres, now chiefly flooded, installed 40 pumps and obtained permits from the Corps of Engineers to pump out the 400,000 acre-feet of lake water and discharge it to the San Joaquin River. Periodic floodings like this can dissolve some of the salts accumulated in the lakebed soils and are quite beneficial to agriculture if they can be removed from the closed basin. These waters, like drain waters from other irrigation in San Joaquin Valley, continued down the San Joaquin River to the bay.

In November 1983 the Kesterson Reservoir made headlines in San Francisco: "When Gary Zahrn saw the newly hatched birds at Kesterson Wild Life Refuge he was reminded of the thousands of deformed babies born in the 1960's after their mothers innocently took the notorious sedative thalybdomine during pregnancy. The birds had stumps for feet, stumps for wings; some were missing eyes and beaks." U.S. Fish and Wildlife Service found in the 1,200 acres of ponds that as many as 90 percent coots and grebes died soon after hatching and that frogs, crayfish and clams had disappeared. The mineral Selenium was found in fish and ducks and was suspected of the mutations and deaths observed.

By 1984 San Francisco headlines were about "Chemical Disaster" and "Poison in The Valley," telling of trees and flowers withering, edozens of cattle emaciated and losing their hair; they had to be fenced off from the toxic water to which they seemed to be attracted like drug addicts (this on the Freitas ranch, adjacent to Kesterson). The disasterous effects of the toxic water upon animal and plant and aquatic life were far greater than could be expected of the Selenium concentrations that were indicated in early chemical analyses for the Bureau of Reclamation. Presser and Barnes of the U.S.G.S. found that part of the selenium is in organic compounds which must be oxidized to selenates (SeO_4^{2}) and then reduced to selenites (SeO_3^{-2}) to derive the total content of Se. Se flowing from the San Luis drain into Kesterson Reservoir (pond 2) on Aug. 3, 1983 was 18 ppb without this refined procedure, increased by oxidation and reduction to 280 ppb, about 15 times as great. (Analyses by U.S.G.S.)

By contrast with the water entering Kesterson from San Luis drain, which ranged from 280 to 350 ppb Se, the water in the lowest of the evaporation ponds (pond 11) ranged from 15 to 60 ppb Se. Where does the Se go in traversing these ponds? If by seepage to ground water, the water from the San Luis drain should not exceed 10 ppb of selenium, the limit for safe drinking water.

As pointed out in Bureau of Reclamation Status Report on "The San Luis Unit Special Study" (August 1984) Kesterson continues to be used as the final disposal unit for agricultural drainage from about 8,000 acres of the Westlands Water District. The concentration of Se at waste entries in the irrigated area ranged from about 1,400 ppb (southernmost) to 200 ppb 10 miles to the north. The Bureau plans to utilize the Kesterson Reservoir and its 8,000-acre toxic-source area intensively for research, including studies of selenium removal and the amount of selenium in sediments and water. There is also urgent need for studies and monitoring wells in the sediments underneath and adjacent to Kesterson, and also in the area eastward to the San Joaquin River.

The B.R. also reports that about 166,000 acres now needing drainage have no place to dispose of their drainage water. The Westland Water District has a total of 500,000 acres of which an unknown proportion contain the coast Range sediments that contain the pyrite that appears to be a host to selenium. These lands with their potential for creating toxic amount of selenium will continue to be irrigated and discharge their toxic wastes directly or indirectly into the Delta and S.F. Bay (and may also market crops that bear excessive Selenium since it readily enters the food chain) unless the State Board of Water Quality Control or the Federal E.P.A. places restrictions on toxic waste discharge.

For consistency of policy throughout California, the State Board of Health Service or the Federal E.P.A. or both may well prohibit the discharge of toxic wastes into the San Luis Drain if it is found that the Kesterson reservoir receiving the outflow of the drain is a recharge area for ground water used for drinking water. This would require containment of the toxic wastes upon the property of the waste-producing irrigator, or the treatment of the water to eliminate the toxicity. This is becoming established practice in Silicon Valley.

According to the Bureau of Reclamation "the loss in gross income in the Westlands Water District alone would reach $25 million if nothing is done to alleviate the growing salt-management problem." Put positively, the Westland Water District stands to gain $25 million if it can solve the problem of selenium management. Thus it stands to profit greatly by disposing of its toxic waste without dumping it upon others--whether by evaporation ponds or storage ponds or pumping to closed basins in the Coast Ranges, or supporting chemical or hydrologic research, or experimenting with sprinkler irrigation, rice irrigation etc. And if the land is finally classified as poisonous and unsuitable for irrigation, it may be profitable to lease the water right to Southern California. Westlands' contract for 1 1/4 million acre-feet annually is 2 1/2 times the amount sent to Metropolitan Water District of Southern California by the State Water Project.

COMMENTS FOR THE SYMPOSIUM PROCEEDINGS

James J. Geraghty

Before coming here today, I thought long and hard about what I might say to you that would be of value. I could of course go on for hours--about unusual ground water situations I've encountered, people with whom I've shared fascinating ground water experiences, or current ground water issues. But we don't have hours available today and so I thought it might be best, for a group of ground water professionals whose education and training have been pretty much along similar lines, to speak about a trend I see that could get many of us in trouble in the very near future. That is the trend toward litigation--a field quite alien to what most of us learned in school.

Litigation is an area peopled by attorneys, judges, hearing officers, juries, plaintiffs, defendants, citizens groups, the press, and insurance companies--all of whom function according to rules very different from our scientific ground water rules. Litigation is a battle--an adversarial proceeding--commonly with large amounts of money at stake. It is an area in which, all too often, no holds are barred. At an increasing pace, problems of ground water contamination are moving into the area of litigation. It is here finally that the hard truths about ground water occurrence and the behavior of contaminants in the earth must emerge--to be fought over and disputed by expert witnesses who are asked to respond to tough questions by attorneys.

It would be well to remind ourselves that attorneys are trained to find the weak spots in an expert's testimony and to discredit him in any way they can--fairly or unfairly. I have watched good ground water people be almost torn to pieces on the witness stand simply on the basis of discrepancies in their typed resumes. In several instances, the attorneys had researched in depth the educational and professional background of a witness--and had found contradictions therein that the witness was forced to admit.

Recently, I was in a case where a top-flight ground water consultant was ordered to limit his testimony to topics that did not involve geology--simply because his education had been in engineering and the court decided that he would not be permitted to testify on hydrogeological matters. In another case, the judge stipulated that the expert couldn't talk about ground water contamination because he wasn't a chemist. It's pretty embarrassing--after you've worked for a long time preparing yourself for an important court case--to be discredited and asked to step down from the witness stand before you can present even one scientific finding.

But your problems may not be over even if you do get accepted by both sides as a qualified expert witness. The next major pitfall may be in the ground water interpretations you make in your reports and testimony. In that regard, I would like to make a distinction between what I'll call hard facts and interpretations made from those facts.

Hard facts are those that can be measured, quantified, and agreed upon according to recognized and accepted scientific rules or principles. Interpretations are what the expert makes from those facts, based on his level of knowledge, the state-of-the-art, and all too often on misconceptions he has acquired and his eagerness to make a complete picture out of a scanty amount of data.

So-called hard facts may themselves not always be totally reliable, but they are far less vulnerable to challenge than interpretations. Examples of types of hard facts in the ground waer field are: (1) locations, diameters, and depths of wells, (2) geologic logs of wells, (3) water levels in wells, and (4) laboratory analyses of soils and ground water samples. A relatively small amount of hard data may be adequate to show whether (1) the geologic formations at a site are unconsolidated or consolidated, (2) the geologic units are flat or tilted, (3) the ground water was flowing in a specific direction at the time of water-level measurement, (4) the aquifer has a low or a high yield, or (5) the ground water does or does not contain contaminants.

But a very large amount of hard data may be needed to prove, beyond a shadow of a doubt, whether: (1) individual geologic units extend laterally between wells, as shown on an interpretive geologic cross section, (2) the contour lines on a water-table or potentiometric map are accurate and the arrows placed on that map truly depict the direction of ground water flow, (3) the contaminated ground water came from the suspected source or from somewhere else, or (4) the direction of contaminant movement has been or will continue to be as shown on the plume map.

In essence, there always will be significant differences of opinion among ground water specialists on how to interpret a given body of hard data. Thus, in litigation, if the expert witness is forced to admit weaknesses in his interpretations or that there could be alternative explanations, judges and juries may simply decide that he's not credible. After all, the essential purpose of litigation is to pit one expert against another, with each side trying to show that the other side's expert should not be believed. Searching for the real scientific truth is not really the objective of this exercise.

We all will find ourselves more and more in the role of expert, required on the witness stand to support with facts every statement we make. We still may get by, at least in some instances, by claiming that we exercised reasonable professional or engineering judgment in arriving at our conclusions. But I suggest that we all had better tighten up our logic if we're to successfully defend ourselves in this increasingly adversarial world.

I have tried here today to give you, in a very short period of time, just a few impressions I have gained about our ground water field. In essence, it is a brand-new field compared with others. Our body of professionals is still far too small to deal with the mushrooming problems. And it is increasingly moving into the courtroom--where you no longer will have the comfort of holding learned scientific discussions with your peers. But that's not all bad news. We are all more in demand than ever. And what could be more satisfying than knowing that we will have far more problems to solve tomorrow than we had yesterday?

MR. WALTON'S SPEECH AT THE KEYNOTE SESSION IN LAS VEGAS

William C. Walton

In preparation for this presentation, I was surprised to remember that my first experience with ground water contamination came thirty years ago in the 1950's. I was assigned to work with C.E. Theis during a three-week period in Ohio. We were assessing the probable impacts of an accidental release of radio-active materials from potential nuclear power plants at several selected sites. We assembled all readily-available information into a database and made aquifer thickness, property maps and water-level maps. We made flow-net and mass flow-rate analyses. We drew flow-lines at right angles to water-level contours and estimated ground water velocities, travel distances and travel times. We also estimated concentrations, taking into account decay of radioactive materials. You might say that we modeled advection and decay with analytical aequations. There were no practical methods at that time for simulating dispersion and adsorption. We had to use slide rules because analogs and digital computers were not available. I have since noticed that many of the simple techniques we used in the 1950's are still usable today, under favorable circumstances. I remember walking along the Scioto River in Ohio with Theis and discussing the results of an induced infiltration pumping test. I stated that I thought my computations were within a few percent of the true answer. Theis chuckled and stated that he thought my accuracy expectations were unrealistic, and he reminded me that ground water resource evaluation is not a precise, highly-accurate science but rather is the science and art of approximation. My experience indicates that prediction accuracies of plus or minus 35% are attainable, most of the time. I have always been suspicious of those who do not round off their predictions and have deterministic rather than range attitudes. My second experience with ground water contamination came in the 1960's in connection with a salt-water intrusion problem along the coast in North Carolina. Heavy pumping, associated with mining, caused fresh-water, salt-water interfaces to migrate toward water-supply wells. I was working with Brachears, Guyton and Harshbarger. We divided the aquifer under investigation into cells and applied the principles of mass balance to predict chloride concentrations. Again, there were no practical methods for simulating dispersion and, of course, with chlorides, we were not concerned with adsorption. I became acquainted with upconing and the possibility of contamination. Vertically speaking, contamination is, after all, not two-dimensional but three-dimensional. We always seem to know the least about the vertical components of flow and contamination. Guyton taught me how to look for clues concerning past historical data and also to be suspicious of every piece of data until it has been thoroughly reviewed. Sometimes, much can be learned by studying the sequence of past historical events and newspaper articles often provide important clues concerning the past. The calibration of a model can be no stronger than the past historical data base. I learned the wisdom of separating technical people from attorneys until the technical aspects of a case have been agreed upon. We were dealing with a multi-layered aquifer system with complex parameters and boundaries and initial conditions and it became apparent to me that there were a large number of possible combinations of parameters and boundaries and initial conditions that would calibrate against past historical data. The un-uniqueness of models became obvious. I have since observed a great deal of unnecessry misery caused by unrealiatic expectations concerning the uniqueness of models. Like everyone else, my experiences with ground water contamination accelerated greatly in the 1970's. Analog models became obsolete, and digital computers became of age. I became acquainted with organic contaminant migration in connection with a study in Western Canada that had to do with coal, fly-ash waste disposal. We were able to simulate dispersion after estimating the parameter dispersivity and adsorption was represented by the distribution coefficient and the retardation factor. We were most concerned with heavy metals. Organic contamination had not yet caught up with society. We began to appreciate the dispersivity, a scaled dependent, and that distribution coefficients are dependent upon many complex conditions. I remember working with George Pinder and Emil Frind on a permanent nuclear waste repository citing project in Eastern United States. The long-term dissolution of salt-beds were being investigated. I became acquainted with modeling techniques associated with diffusion and flow in sub-outcrop areas of salt-beds. Although we used sophisticated, finite element models, the foundation of our work was still low in mass balances. I was impressed with the dependency of mass transport models on flow models. In the 1970's, I became familiar with fortran 4 and basic computer languages. Thanks to the efforts of Tom Prickett and Phil Davis, computers no longer intimidate me. I have always thought of computers as a work horse and not as a substitute for my logic and reasoning. The 1980's could be appropriately labeled the decade of the organics. Thanks to Geraghty & Miller, I had the opportunity of defining many large organic plumes. I became familiar with most of the original 129 priority pollutants. I got acquainted with octanol water partition coefficients, organic carbon contents in designing remedial measures we were concerned as much with desorption as with adsorption. Most models assumed desorption as a mere image of adsorption which we now know is unrealistic. The most difficult problem associated with designing remedial measures is estimating the amounts of contaminant mass in the liquid and solid phases. Adsorption and desorption distribution coefficients can lead to approximations of contaminant masses temporarily stored in ground water systems. I noticed that retardation factors are used to slow the velocity of contaminant migration. However, some people fail to use the retardation factor in determining the amount of mass in the liquid and solid phases when predicting concentration levels. Thanks to Mary Anderson and John Cherry, we now have a better appreciation for all of the complexities and limitations associated with modeling adsorption and dispersion. Surely after reading their papers, no one could have unreasonably high expectations concerning the accuracy of mass transport modeling estimates. And what can we expect in the rest of the 1980's and the 1990's? I hope we pay more attention to the basic principles of geology and hyrogeology and geochemistry. I hope we are able to reduce the uncertainties surrounding dispersion and adsorption and desorption. I hope field tests for determining dispersivities and porosities and in-situ distribution coefficients become as commonplace as pumping tests are today. I hope that computer programs become more friendly and readily available. I hope that micro-computer software or flow of mass transport and graphics are widely distributed. I hope we all learn to use models that are in tune with data bases and levels of decision. I hope we realize that simple analytical models are useful in many cases. Finally, I hope that all of you are rewarded as much as I have been rewarded while laboring in the ground water field.

In conclusion, I have done so much with so little for so long. Now, I can do anything with nothing and a computer.

COPING WITH GROUND WATER UNCERTAINTIES -- 1940-1990

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History

There is nothing special about the years 1940 or 1990. I merely note a span of years to show how our profession is changing. We tend to get so busy with our work that we seldom pause to reflect on the philosophies that are related to objectives of studies and work in ground water. My purpose is to note some changes through the years -- some changes in human needs, in objectives, and in methods of study.

Prior to 1940 the subject of ground water was seldom discussed. The Ground Water Branch of the U. S. Geological Survey had several reconnaissance ground-water projects on a regional basis, the emphasis being on finding productive aquifers. There was very little test drilling, and pumping tests on wells were rare.

Between 1940 and 1960 the U. S. Geological Survey expanded its studies and had cooperative investigations in most states. After 1950 the quantitative aspects of ground water received increasing attention, and aquifer performance tests were getting to be standard procedure. Most of the work on water quality related to salt-water intrusion in coastal aquifers and to interior areas where salt-water aquifers underlie fresh water aquifers. Only a few colleges taught as much as one course in ground water.

Interest in ground-water quality was sparked about 1960 by noticeable but scattered contamination from septic tank systems and by awareness that dispersal of radioactive wastes was a problem. Both the intensive studies and philosophical approaches on radioactive wastes by the Atomic Energy Commission and its contractors, as well as by the U. S. Geological Survey, provided a good foundation for studies of hazardous chemical wastes. Only a few states had a program for coping with changes in ground-water quality before 1970.

In the ten-year period following 1970, the newly-formed U. S. Environmental Protection Agency gave increasing attention to waste management practices and to ground-water contamination. It was difficult for the Agency to get overall perspective of the problems. A question frequently asked was "Is ground-water contamination really a problem?" The Agency moved forward with strict regulations, which in many cases were not compatible with the variability of real hydrogeologic conditions.

Systems for evaluation of waste sites were developed, but there was no consistent acceptance by regulatory agencies and waste management officials. Observing the wide range of hydrogeologic settings and variability in types and in behavior of wastes, many hydrogeologists started adopting the theme that "all waste sites are unique, and, therefore, all sites need special intensive studies without regard to use of past experiences".

In the past ten years, many research projects have been initiated on the behavior of contaminants in the ground. Many of the projects have focused on the use of transport models, requiring mathematical formulations and considerable data. Massive data bases are built where intensive studies are made of waste sites. Whereas a ground-water hydrologist in 1940 may have been pleased to have one test well per square mile, a ground-water hydrologist today may not be satisfied with 30 test wells around a specific waste site.

Suggested Future Objectives

Introduction -- Much work is now being directed to determine the behavior of contaminants in the ground both in general studies and at specific places. Considering the range of subjects relating to the behavior of contaminants in the ground, it is normal for some subjects to be underemphasized while others are overemphasized. The application of mathematical approaches has had fruitful results in recent years, but the approaches have now far outrun data evaluation of hydrogeologic conditions. Following are some suggested future objectives that deserve increased attention.

Description and Evaluation -- It is unfortunate that we do not have a consistent or standardized approach to describing and evaluating existing and potential ground-water contamination sites. Many workers object to attempts at evaluating and comparing sites. Objections to site evaluations are based partly on the "uniqueness of site" syndrome and the fear that a site evaluation system would be relied on unduly by regulatory agencies and thereby hamper specific intensive studies.

Efforts to compile case histories of ground-water contamination are limited because certain key elements of hydrogeologic settings are almost always omitted. A concise standardized site descriptive format is needed for every study, no matter what the main emphasis of the report may be. We cannot make optimum use of our past experience if we do not have a site descriptive format. Such a descriptive format requires little effort, but without it our science is hampered and waste management officials may not be able to clearly defend their positions on waste sites.

Unresolved Issues -- Recognizing and assessing some unresolved issues represent the first step in improving our protection of ground-water quality. Some issues, expressed in question form, that both the U. S. Environmental Protection Agency and most state regulatory agencies have not fully considered, are listed below.

1. Should there be exclusive and complete reliance on laws and regulations as regards water and waste management? Where bound by regulations that do not equitably apply, should boards or commissions make appropriate discretionary decisions?
2. To what extent should ground-water contamination be tolerated? Should subsurface areas be zoned -- some for residence of contaminated water and others for water supply? Should subsurface trespass of contaminated ground water be unacceptable in some cases and acceptable in others?
3. How should regulatory agencies cope with latent movement of contaminated ground water into streams and lakes?
4. How should regulatory agencies cope with existing faulty waste situations that cannot comply with new or recently developed regulations?
5. Should complete containment and concentration of hazardous wastes in the ground be required? To what extent can dispersion and dilution be relied on?
6. Is an overall policy needed as to placing waste sites in recharge areas? In discharge areas?
7. How reliable are deterministic transport models under variable data bases and ranges of hydrogeologic conditions? How suitable are probabilistic approaches to predicted contaminant behavior?

Information Framework and Data Base -- Everyone agrees that we need to know more about the behavior of contaminated water in the ground in specific cases. How precise do we need to be in our predictions of movement of contaminated ground water? The specific features of some regulations and the pressures of litigation almost mandate precise data collection and accurate predictions of contaminant movement in many cases. Specialists in waste management practices are geared for massive data collection, partly because data collection may be their livelihood and partly because they feel that replacing some data collection with strong inferences would be less reliable than complete data coverage. Reliable and good inferences, giving a high plateau of knowledge at an early stage of study, can be made without large masses of data in many cases. Yet, there is a prevailing trend not to accept conclusions unless they are supported completely with data. It is proper to look beyond this trend to an informational framework that includes both key elements of data and appropriate inferences based on past experiences.

HISTORICAL PERSPECTIVES -- PAST, PRESENT and FUTURE

(Summary)

John W. Harshbarger

History

For much of the past 50 years, the driller's approach was to drill holes by cable tools and install slotted mills knife casing. Well logs were very crude, commonly of bar stool vintage. Most drillers had little or no use for engineers, geologists, and others. Technical persons were not well informed on hydrogeology and quantitative aspects; and subsurface data were considered useful for drillers only. There were differences between engineers and geologists, and there was not much application of quantitative approach except for the U.S. Geological Survey and several State agencies.

My early experiences were based on academic engineering and geology; short-term pump tests were usually 6-8 hours per day. There were virtually no borehole geophysical logs and/or surficial study of the aquifer area for analysis of data. Theis and Theim equations were used. Basic concepts and quantitative equations for ground water systems were given by Theis in the mid-1930's. Basic geological and water components were obtained from the Johnson Water Well Handbook (1947).

Many texts have been published since the late 1950's. Several authors were Todd, Walton, Davis & DeWeist, Ven te Chow, Freeze & Cherry, (Oil Well Reservoirs, Mechanical Hydraulics), Neuman, U.S. Geol. Survey, Ferris, and others. Also, there are many publications from foreign countries. Additional texts are published every year and many new equations and formulas for non-steady state systems developed. The hydrologic analog model was developed by H.E. Skibitzke and others, and today digital and numerical models are common place. Formal education and training in schools and universities were encouraged by the U.S. Geol. Survey and University-Council on Water Resources (1960) and 10 to 15 universities established formal degree programs, short courses, and study programs.

Subsurface geologic models were based on improved quality of geologic mapping and drill cuttings from driller's logs. Flow net analyses were improved with drilling of many more wells, more precise recording of water level data, collection of water samples, and rotary drilling techniques. Important new information was obtained from borehole geophysical logs which enhanced knowledge on aquifer media as related to T and K values on nonhomogenous rock units. Drillers and groundwater technical experts worked more closely in the exchange of knowledge to improve drilling methods, well design, optimum use of rotary equipment, utilization of air foam and other procedures. Well development improved and enabled water supply development in areas of rock sequences not tapped previously. The enhanced knowledge of geological structural conditions coupled with the improved techniques were basic in development of water resources in volcanic tuffs, large sandstone bodies, and fractured rocks.

In Senator Goldwater's first term, he ordered the U.S. Geol. Survey to find water for the City of Flagstaff. A wellfield was located in a fault zone overlain by volcanic rocks. Other areas of groundwater development in fractured rocks in which I was a professional advisor included altiplano areas in the Andes in Peru and Chile, the New Valley in the Sahara Desert, Saudia Arabia, Jordan, Lebannon, North Africa, Egypt and Nigeria. These wellfields provided excellent groundwater reservoirs which yield large amounts of water in arid deserts.

The salinity problem in the lower Colorado River, United States and Mexico in the Yuma-San Luis, Mexico area comprised an international problem in the early 1960's. New concepts and comprehensive study of the cause and effect relationships provided a solution for both countries. Sophisticated modeling techniques provided a basis of common agreement for expanded pumpage in both countries.

Today, there is a need to develop groundwater management operations to meet demand and restrictions by water management agencies. The most recent concern pertains to water quality and contamination. Professional persons must be equipped to provide impact analyses of withdrawal effects on aquifer systems. Such predictions may include future effects that will occur in 100 years in places; for example, the Arizona Groundwater Law of 1980 has requirements for prediction analyses. Contamination in shallow ground water systems poses immense problems today and in future years.

In regard to the future, several items are certain, we will have to pay more taxes and the cost of water will be greater; there will be more restrictions and laws on water use and management; and additional ground water supplies, need to be discovered and/or developed in the southwestern United States and other arid areas.

Are There Additional Sources of Water?

A. Northwest and Canada--Water transport problems to include construction of grandiose conveyance systems may be considered when the price of water can be agreed upon by both the consumers and water right holders. What is the ultimate price of water, is it comparable to the price of bourbon?

B. Deep Groundwater Basins (2,000 to 5,000 feet)

1. Vast reservoirs may occur under state and federal lands, military, forest, Bureau of Land Management areas, and private lands which have not been tapped for exploitation.
2. Additional legislation and new laws will be needed for transfer and development when warranted by price controls.
3. Drilling equipment to penetrate 5,000 feet or more will be needed along with improved construction methods and appropriate pump equipment. The application and utilization of oil-well technology for such development would make such deep well drilling feasible. Recent estimates indicate that 4 to 5×10^9 acre-feet or about 2,200 cubic kilometers are stored in alluvial basins and volcanic rock sequences in Arizona.
4. It will be necessary to educate and advise federal and state legislators to exercise imagination and to revise legal and managerial regulations to enable utilization of such water reserves.
5. Sophisticated models for quantitative analysis and water quality analysis are now available for utilization, but they are not being applied to current ground water problems to maximize the capabilities of this technique.
6. The technical knowledge and well performance are known and tested. It remains for the legal, political, and management areas to upgrade their knowledge and expertise on the nontechnical aspects required for utilization of groundwater reservoirs for optimum public use.

The ability of professional people to pool resources and information in the scientific community may well be a major aspect of management of our vital water resources in the future.

REFLECTIONS ON THE SCIENCE AND TECHNOLOGY OF GROUND WATER

Raphael G. Kazmann, P.E.

The Situation Early On

Modern ground water hydrology, and hydrologic engineering had their beginnings in 1935, with the publication of C.V. Theis's non-equilibrium formula (Theis, 1935). In 1942, Wenzel's U.S.G.S. Water Supply Paper 887 included a table of the well function that, for the first time, enabled engineers to readily make numerical computations of discharge, drawdowns, interferences, and similar matters. After WWII, theoretical studies of laminar flow systems made by Jacob, Theis, Lohman and others including Mahdi Hantush, whom we honor today, placed into the public record theoretical solutions to many problems, some which had already been met in the field and others that they dreamed up and may yet be encountered.

By 1963 there was a great number of papers containing Theis-inspired solutions to ground water problems contained in a great number of scientific periodicals. The Dept. of Civil Engineering of Colorado State University then sponsored a conference on transient ground water hydraulics. The topics presented included the use of analog models and some mathematical models for use with digital computers. Most of the problems of a purely potentiometric nature encountered by the engineering profession were discussed in the proceedings volume (Maasland and Bittinger, 1963) and the theoretical solutions may be found there. When the term "potentiometric" is used in this paper, I mean that the water levels in tightly cased, completely penetrating wells can be used directly in the analysis without correction for density. The subject matter was updated by Glover (1974) and since then, for all practical purposes, the theoretical answers to potentiometric problems are available in these two reference books.

Simultaneously so successful was the application of mathematical models to engineering problems that no hydrogeological study for practical ends is now considered adequate if a mathematical model (or an analog model) has not been used to obtain the required answers. This has had many good results and many unforseen side-effects (we will return to them later).

The Situation, More Recently

Sometime between 1963 and 1970 murphy's law began to make itself felt. You know, the one that says, "Whenever a system becomes completely defined some damn fool discovers something that either abolishes the system or expands it beyond recognition." The earliest signal that the law was operating were the problems involved in sea-water intrusion - these could not be handled by purely potentiometric methods because of the variable densities observed in the transition zone between the sea water and fresh water.

These problems were, however, confined to the coastal areas and, although important, did not affect most of the practitioners of ground water hydrology or engineering. Another straw in the wind was Jones's (1969) emphasis on geochemical hydrodynamics which pointed out that clays can act as semi-pervious (osmotic) membranes. Such clays, previously thought to be impervious, permit movement of fresh water into salt water. Practical problems involving these matters could be solved by including the potentiometric components as part of a larger solution. The trouble was that the mathematics was no longer clear cut and the site-specific constants (analogous to T and S) were hardly definable, much less to be produced by a test at a specific site. A certain indeterminacy was creeping into the mathematical framework that was being applied, although to the outside world, all was well.

In the early 1970's attention was turned to the use of aquifers for purposes other than water supply: cyclic storage of fresh water in saline aquifers, possible use of saline aquifers for the storage of industrial wastes, and industrial storage of heat (or heated water) in aquifers. Toward the end of the decade the use of aquifers for domestic water-source heat pumps became a real option following the improvement of the seals of the compressors. The potentiometric solutions, it should be noted, were also modified to solve problems of mine drainage, excavation of deep foundations, artificial recharge, and in circumstances where the thickness of the aquifer and the storativity of the aquifer were not constant. During the same time frame, starting after 1975, the problems connected with the protection of the environment such as those of leachate from sanitary land fills and land disposal of petrochemical wastes became prominent. Here, again, the purely potentiometric methods were applied, but with mixed success.

In a nutshell, potentiometric methods are methods that, in effect, "homogenize" aquifers. You obtain averages directly from measuring water levels in monitor wells while stressing the aquifer by pumping a well. These averages of transmissivity and storativity are used in the modelling process.

In the new set of problems the assumption of homogeneity is wrong. Essentially you are trying to trace the movement of a foreign substance in an aquifer. In such instances the micro-structure of the aquifer governs: permeability lenses, deposits or lenses of greater or lesser silt dissemination, stratification, various types of clay and silt, and localized, non-uniform boundaries. But most important: one particle of fluid is not the same as every other particle: there are density and viscosity differences to take into account. In the more complicated instances, there are interactions between the fluids and the aquifer materials, chemical and physical and sometimes bacteriological. Most of these reactions cannot be modelled at this time (1984) simply through lack of knowledge. Unfortunately, we have a number of researchers who claim that they can incorporate all of these additional parameters into existing mathematical models. And the outside world, including our legislators, believe them. Hell, they can't even incorporate dispersivity into their models properly and most of them don't even understand what dispersivity is. In the real world silty sands, silts, and even clays, possess permeability that must be taken into account. And, much to our surprise, clays are not impermeable to the movement of complex hydrocarbons. Certain hydrocarbons alter the fabric of the clay, making it pervious. Moreover the movement of such "foreign" substances in many instances are only slightly influenced by potentiometric gradients--the governing forces may well be density differences and differences in viscosity which cause "fingering." In many instances clays and silts "bind" or "fix" the complex hydrocarbons and reduce the potential pollution of aquifers.

To sum up: Potentiometric problems can be solved as accurately and reliably as any other engineering problem. We can predict drawdowns, interferences, pumping water levels, dewatering layouts, and impacts of ground water offtake under a variety of hydrogeological conditions. Given the funds and time (and these depend on how precisely you wish to define the system), the problems are solvable. An approximate solution is inexpensive, the more precise and accurate the required answer is, the higher the cost--disproportionately higher.

Problems of miscible displacement and the travel of foreign fluids in aquifers are far more expensive to solve and, in many instances, we are still unable to set forth a completely satisfactory solution. This does not mean that we cannot come up with engineering answers but it does mean that we cannot produce a reliable, useful mathematical model based on a few (or many) field tests.

Now What?

What of the future? My crystal ball is as unreliable as anyone's, but given that caveat, here goes:

The principal obstacle to the systematic development of ground water evaluation and application, and this includes protection of potable aquifers from contamination and degradation from any source, is the Congress of the United States! Consisting primarily of lawyers (and I have a high regard for attorneys, in fact my daughter is a practicing attorney), and motivated by a desire for reelection, members of Congress have a tendency to disregard the laws of nature and to try to achieve, by fiat, what they think is beneficial and will add to their popularity. They seem to think that all engineering and scientific answers are at hand and all that is necessary is to throw money (and regulations) at a generic problem and Lo! it will be immediately cured. And while their motives are most laudable, good intentions are not enough. Before you regulate something, or someone, you have to understand the context and weigh the cost against the benefits. This has not been done.

For example, one of the most ridiculous themes of modern times is the goal to restore all of our surface waters to their pristine, Pleistocene, condition. It reminds me of the story, probably a canard, that one House of the State of Tennessee passed a bill requiring the value of PI to be three. However, the story goes, the other House defeated the Bill. Unfortunately the U.S. Congress is not as wise. The thermodynamics of life, not to mention cost, make the restoration of the land and water to its assumed Pleistocene condition a chimera. And when a nation seriously tries to achieve the unachievable, it is flirting with disaster.

This tendency to legislate a clean environment, including protection of aquifers, is laudable, but full of hidden fish hooks. Its primary effect is to freeze laws and regulations to the technology of the moment that the legislation is written. We know that the detailed regulations are not written by Congress, they are prepared by another group of lawyers and published in the Federal Register at which time they become law. Those technicians charged with enforcing the regulations are only doing their job even if the net result is a waste of money and of no measurable benefit to the environment and the population. The regulations are predicated on the reliability of the methods for the detection and accurate determination of contaminents, chemical and bacteriological analyses, the extrapolation of the results of animal experiments to human beings, and the attribution of increases in certain dread diseases such as cancer, or cardio-vascular failures, to "environmental" pollution. Daniel Webster said it as well as anyone: "A strong conviction that something must be done is the parent of many bad measures."

When it comes to ground water protection, we have the Superfund, primarily for cleaning up existing petrochemical waste disposal pits, and a multitude of other ground water protection measures. There is no inkling of an evaluation of costs and benefits in any of this. The legislation is based on what might be termed "Water Pollution Panic." At this point we might well ask, "What is pollution?" I doubt that anyone will agree on a single answer but let me submit that water or land is "polluted" when the user has to spend money to make it usable for him. If the quality does not interfere with his use, treatment free, it is unpolluted as far as he is concerned. Mississippi River water is "polluted" if it is to be used for the making of cigarette paper or steam for high pressure boilers or for drinking purposes. It is unpolluted if used for navigation or crop irrigation or the cooling of thermo-electric generating plants.

It is well to gain some perspective on the importance and significance of environmental pollution. Figures have been bandied about that environmental pollution is the cause of, possibly, 80 percent of all major ailments and that genetic weaknesses and an unaccounted for residue of causes can be blamed for the other 20 percent. When we look to see how these figures were obtained, another pictures appears: the scientists compared the incidence of stomach cancer, to make up an example, in Finland and in Kenya, the Finnish rate might be, possibly, 10 times the rate in Kenya. So if the Kenyan rate is considered the minimum, then the Finnish rate must be due to environmental causes--and these causes include the consumption of smoked fish, smoking drinking, etc., etc., not just the atmosphere and water. The original reports used the word "environmental" to include the personal environments of individuals, not solely the external air, land, and water. If you remove the personal environments - the life styles - from the statistics (and this has been done), you come up with a number between 3 and 5 percent of the major causes of death that might be due to what we call environmental causes. On a cost effective basis, public moneys might better be spent on the 75% than on the 5% (and this is heresy): on a reduction in consumption of refined foods (sugar, flour, etc.), alcohol, and over-the-counter as well as illicit drugs.

Deductions

When it comes to ground water, the threats to the supplies are more potential than immediate. If we confine our discussion to granular materials where movement of ground water is Darcian, then any contaminant that is moving as the result of potentiometric gradients can be halted by creating a zone of zero potentiometric gradient. This is accomplished by proper placement of injection and withdrawal wells (Kimbler, et. al. 1974, pp B-1 through B-6). By so doing, the local people will buy time to devise a method to pump out or neutralize the noxious substance without having to take hasty, expensive, and possibly futile, measures. Where clay layers exist, and the contaminants are dense, chlorinated hydrocarbons that move primarily on the basis of density differences (these foreign substances are not particularly soluble, the maximum solubilities usually being on the order of 200 to 800 ppb) they are "fixed" by the clays whose permeabilities are altered when they come into contact with them. The extremely high concentrations of these substances found in water samples are misleading: these are mixtures, not solutions. Moreover, recent research has shown (Wilson and McNabb, 1983) that there are bacteria in the ground that can consume these hydrocarbons. It is very probable that bacteria can be produced that can be injected into an aquifer and clean up the aquifers at a fraction of the expense of the measures now used.

The standard approach is based on measurements of the potentiometric gradient on the assumption that the contaminants are moving down-gradient. In too many instances curing the problem consists of digging out the wastes (bleeding the volatile contaminants to the atmosphere) and then burying them somewhere else or, possibly, burning them. All of this is done in an atmosphere of extreme urgency, lawyers talk learnedly about impermeable membranes, underlain by clay layers, underlain by sand drains, underlain by a clay layer with monitor wells up and down gradient and perpetual sampling of the fluid in the drains, impervious subsurface walls, etc. etc. etc. ad nauseum. These are all requirements built into one regulation or another for use throughout continental United States: in limestone terranes, glacial till, sandstone, basalt, coastal clays, granites, etc. etc. as though a single regulation could serve to protect the public against systemic poisons, chlorinated hydrocarbons, selenium, chromium, barium, pickle liquors, and the numerous waste products not subsumed in this list. In addition, to complicate things still more, in the concentrations deemed to be deleterious in water, we find that selenium and chromium (in certain forms) to be essential trace elements for human health, not to mention zinc and other trace substances.

The promulgation and the enforcement of such standards, willy nilly throughout the country, will not produce the favorable outcomes that you and I desire. However the mere existence of such laws will serve to stifle innovation and increase the engineering and legal costs of doing anything new. It will stop the use of aquifers for anything but water supply--ground water heat pumps will be hedged with so many needless restrictions that they will be too expensive for residential use. Deep well disposal of wastes, should the well necessarily go through a potable, or potentially potable, aquifer en route to the target aquifer, will be prohibited. The resulting increase in costs will not be seen as an increase in taxes, but the effect is the same--the cost of everything will be made more expensive and it will be harder for the average man to make ends meet. Moreover, the environment will not be made appreciably safer than it would have been if a less hurried, less bureaucratic, approach were taken. It is not the ends that I take issue with, it is the means now promulgated: they are both costly and not effective. They are becoming part of the problem, not the solution. And, as I stated, the Congress of the United States is the source of the trouble.

Conclusions

The future of ground water technology is not assured. Progress is being stifled by the Congress of the United States which, with the best of intentions, is freezing the practice of ground water engineering at the present level of knowledge and technology. This has been accomplished by their misapprehension of "environmental" hazards and their frantic efforts to cure this allegedly dangerous situation. Instead of focussing on the public health they worry about the "environment." The laws are so poorly written that, for example, the owners of safest public water supplies in the world are forced to spend large sums of money to install activated carbon filters to remove traces of what may be carcenogenic materials such as choloroform from the water. The money removed from the consumer by the forced addition of added water treatment could be better spent by the consumer on his diet. The Congress seems to have no confidence that a community will strive to produce safe water without a national law mandating this. Such an attitude treats responsible citizens as backward children. In the real world, in the absence of laws, mistakes are made and if they are painful others learn to avoid them. In fact we have institutionalized this process by investigating airplane crashes and car accidents to find out what has been done wrong. It is far less costly to society to learn from accidents than it is to try to prevent accidents by passing laws that stifle innovation.

Similar wasteful expenditures are found in the areas of waste treatment, where often the treated effluent is, by every standard, more drinkable than the water into which it is being discharged--but all must be treated alike whether the receiving water is the Mississippi River or Alum Creek. This is bankruptcy in the name of an arbitrary fairness.

Deep well disposal of wastes is on the road to being reduced drastically because of some unproven threat to water supplies--the correction of a temporary break in the well would be far less expensive to society than the nationwide abandonment of a technically sound, effective program of waste injection.

The movement of certain chlorinated hydrocarbons through the bottoms of land fills has caused panic in certain agencies. Nevertheless, one would be hard put to discover any damage to the public health due to the movement of such wastes. Even the Love Canal incident, which did not cause an outbreak of disease, would have been avoided if the local School Board, in its ignorance, had not sold the land fill for use as residential property.

As I have already pointed out, clays have an ability to halt the movement of many hydrocarbon wastes. And if a waste plume is found in an aquifer, it is not difficult to create a zone of stasis in the aquifer while corrective measures can be carefully planned and accomplished--there is no need for haste or panic.

Thus we must reach the conclusion that the increasing involvement of the Congress of the United States, in the name of improving the health of the public, constitutes the principal threat to technologic progress, with no persuasive showing that the health of the population has improved, or will improve, due to these efforts.

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Session I
Dealing with Underground Storage Tanks

DEVELOPMENT AND IMPLEMENTATION OF REGULATIONS TO CONTROL UNDERGROUND FUEL STORAGE TANKS ON CAPE COD

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Background

Cape Cod is a peninsula extending nearly one hundred miles seaward into the Atlantic Ocean from the mainland of Massachusetts. The sole source of drinking water is a glacial aquifer largely comprised of sand and gravel outwash. Approximately 130 public supply wells and some 15,000 private on-lot wells furnish nearly nine billion gallons of water per year to the residents of the fifteen Cape Cod towns.

Since the 1970's ground water protection has been a well recognized issue to Cape Cod officials. Early efforts by the Association for the Preservation of Cape Cod and the Cape Cod Planning and Economic Development Commission (through a series of EPA-208 grants) precipitated an active ground water protection program at the County level.[1] This program consists of ground water monitoring, town zoning by-laws, general town by-laws, and town board of health regulations.

The problem of leaking underground storage tanks was identified as a major threat to ground water in the 1978 Water Quality Management Plan/EIS for Cape Cod published by the Cape Cod Planning and Economic Development Commission.[2] About the same time a gasoline spill from a ruptured underground storage tank at a service station in the Town of Truro was discovered. After ground water investigations were completed, a plume of gasoline-contaminated ground water was determined to be migrating toward a nearby public water supply well field. Much of the product floating on

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[1] The area referred to as Cape Cod in this paper is synonomous with Barnstable County which comprises fifteen towns.

[2] This document was prepared under Section 208 funding provided by EPA.

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the water table was removed with the use of scavenger wells. A carbon filtration/aeration treatment system has been designed to renovate the remaining ground water in the area which has been contaminated by the dissolved fractions of gasoline. Presently, the estimated cost of cleanup operations is approximately five million dollars.

This incident, as well as others reported nationwide, prompted the Cape Cod Planning and Economic Development Commission to develop a model regulation/by-law to control underground fuel storage tanks. Lacking any legislative authority at the County level, regulations were drafted for enactment at the town level. In Massachusetts, towns have broad home rule authority to protect the public health and welfare through town meeting action to adopt general by-laws.[3] Town boards of health also have far-reaching powers to promulgate any reasonable regulations to protect public health.[4]

The Regulations

Since 1979, all fifteen Cape Cod towns have adopted either general by-laws or health regulations to control underground fuel storage tanks. In either case, the town board of health is the administrative authority which carries out the regulations in conjunction with the fire department which also has some regulatory authority over underground fuel storage. The major provisions include:

(1) Registration;
(2) Inventory control;
(3) Leak testing; and
(4) Specifications for new installations.

Registration of Existing Tanks

The regulation provides that all tanks in excess of a specified size must be registered with the board of health. The minimum size criteria is optional and varies between communities. Some communities have chosen to include residential tanks, while others only focus on commercial size tanks such as those found at gasoline stations. Registration information required to the board of health from tank owners includes the size, type, age, location and material stored.

Inventory Control

The regulation requires that all tanks have a method of accurately gauging the volume of the product, as well as a method of metering removal of the product. Daily records should include a computation of net gain or loss. Periodic inventory verification is required to determine any discrepancies between expected volumes and measured volumes.

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[3] Chapter 40, Section 21 of the Massachusetts General Laws.

[4] Chapter 111, Section 31 of the Massachusetts General Laws.

Leak Testing

Tanks which are 15 years old or older are required to be tested annually using the Petro-Tite (formerly called Kent-Moore) pressure test or any other pressure test providing equivalent safety and effectiveness as approved by the board of health. This test costs approximately $300. The model regulation also requires that non-conforming steel tanks be removed from the ground at 20 years of age and inspected for leaks. If a leak is apparent, an investigation of the amount and location of spilled substance is undertaken at the owner's expense.

Installation of New Tanks

The model regulation requires that new tanks which are installed be of an approved design and protected from internal and external corrosion. Some examples of approved tanks include all fiberglass construction or steel with bonded fiberglass or enamel coating. The installations are to be performed by a manufacturer's approved installation contractor.

Another provision within the model regulation deals with the proximity of underground tanks to water supplies. Cape Cod communities have opted for a number of different approaches in this regard. Some have prohibited any underground storage tanks within a specified distance (2,500 feet is recommended) from a public or private well. Other communities have allowed only fiberglass tanks within these specified distances.

TABLE 1

Town	Total Number Of Tanks Registered
Barnstable	160
Bourne	143
Brewster	39 Commercial 119 Residential
Chatham	56
Dennis	109 Commercial 5 Residential
Eastham	88
Falmouth	No registration requirement.
Harwich	50
Mashpee	0
Orleans	11 Commercial 105 Residential
Provincetown	57
Sandwich	0
Truro	27 Commercial 3 Residential
Wellfleet	44
Yarmouth	
	1,016

Experience to Date

In 1984, the Cape Cod Planning and Economic Development Commission conducted a study to determine the status of underground fuel storage tank regulations Cape-wide. The purpose of this study was to monitor implementation and provide additional assistance to towns in need of improving their regulatory programs. This study involved initial research by the Cape Cod Planning and Economic Development Commission, including examination of local records and interviews with appropriate local officials. The results were tabulated, and along with conclusions and recommendations, presented to local officials at two well attended region-wide workshops.

In conducting the study, the Cape Cod Planning and Economic Development Commission sought to evaluate implementation of each of the major provisions of the regulations including: tank registration requirements, analysis and submittal of inventory control records, leak testing enforcement, and design standards for new tank installations.

Tank registration is required by fourteen of the fifteen towns. Several of these towns require only commercial tanks to register, while others require both commercial and residential tanks to register. Table 1 lists the number and type (commercial/residential) of tanks registered by town.

Through interviews with local officials, it became apparent that this first, and crucial step in the regulatory process, is the most difficult and problematic. Identifying the complete inventory of tanks town-wide and determining specific information such as the tank age and location is, in many cases, a struggle for health agents and fire chiefs due to their widespread nature and a poor information base. Officials often rely on the following sources of information to identify tanks:

(1) Telephone books;
(2) Fire department permits;
(3) Gasoline sale permits (selectmen); and
(4) Driving around to locate sites where tanks may be in use.

These procedures are time-consuming and not always effective. Existing records, such as local gasoline sale permits (selectmen) or fire department tank permits, are scattered and do not always provide the needed information on tank size, age and location in one place. It has been difficult, if not impossible, for local officials to effectively use these (would be) sources of information.

By reviewing current practices used in identifying and registering tanks, it was determined that several classes of underground storage tanks remain unregulated relative to large, easily identified commercial gas stations. These unregulated classes include thousands of residential heating oil tanks and commercial oil and chemical storage tanks.

The vast majority of registered tanks are used by commercial gasoline sales operators. Most town regulations/by-laws require registered tank owners to submit verification that inventory control methods (usually dip-sticking) are being performed. Few towns currently receive and review quarterly statements from tank owners, preferring instead, a method of spot-checking the inventory records during random site visits. The reasons commonly given for this adjustment in regulatory procedure are 5

(1) A lack of staff time among health agents and fire chiefs for review of submitted reports; and

(2) An assumed incentive among tank owners to voluntarily check inventory for product loss on a daily basis as part of good business practices.

Interviews with these officials suggested that the quality and frequency of daily inventory checks is difficult to quantify for all registered tanks.

The Cape Cod Planning and Economic Development Commission identified significant problems with actual daily inventory procedures/methods when making random site visits to local gasoline stations. These visits revealed that current practices at the stations are poor and not reliable, except in the case of detecting very large leaks. It was easily apparent that smaller leaks which may lead to major spills underground would be easily masked by a variety of sources of error. Observed sources of error included

(1) Misreading the dip-stick. (Gasoline will evaporate quickly on the stick, making accurate readings doubtful.

(2) In several cases the sticks, themselves, were badly damaged through careless practices or misuse.[5]

(3) Record keeping was observed to be sloppy and incomprehensible.

A small error in reading the stick or in recording, can lead to a record of large losses or gains in product. Since losses and gains are common due to errors, tank operators may become complacent about record keeping and possible leaks.

Since local regulations were first adopted in 1978, required periodic tank tightness tests have resulted in the discovery of 39 leaks, with detection occurring soon enough in these cases to prevent significant pollution. (See Table 2) To assess implementation of the tank testing requirements by town, the Cape Cod Planning and Economic Development Commission compared local records indicating which tanks were scheduled for inspection in 1983 (15 years of age and older), with the number of tanks on which a test was actually performed. The results of this comparison are listed in Table 3. County-wide, 86% compliance with testing requirements was achieved. Six towns recorded full compliance in 1983 and in only two towns were there substantial difficulties (50% compliance or less) in completing tank tests. In most cases, the owners or operators of underground tanks were very cooperative in complying with the regulations. Many recognized the threat to ground water and were aware of the potential liabilities involved with ground water cleanup.

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[5] In one instance it was difficult to determine the original length of the stick as it appeared to be chewed off.

TABLE 2

Town	Number of Leaks Detected Since Adopting Regulation/By-Law
Barnstable	12
Bourne	8
Brewster	0
Chatham	2
Dennis	0
Eastham	0
Falmouth	-
Harwich	11
Mashpee	0
Orleans	2 Commercial
Provincetown	0
Sandwich	1
Truro	1 Commercial
Wellfleet	2
Yarmouth	
	39

TABLE 3

1983 TANK TIGHTNESS TEST ENFORCEMENT

Town	Number of Tanks Scheduled For Testing - 1983	Number of Tanks Tested - 1983
Barnstable	45	45
Bourne	20	10
Brewster	22	22
Chatham	28	12
Dennis	46	45
Eastham	-	-
Falmouth	-	-
Harwich	10	10
Mashpee	0	0
Orleans	14	12
Provincetown	1	1
Sandwich	0	0
Truro	10	10
Wellfleet	12	12
Yarmouth	-	-
	208	179

A number of tank owners and operators (primarily large, corporate franchises) voluntarily replaced their tanks when faced with the annual testing requirements on a 15-year old tank. They felt it was prudent business practice to keep their underground storage facilities upgraded to the point where testing was not required.

There are several important findings about periodic testing that reinforce its importance. Faced with poor methods and the subjectivity that exists with interpreting daily inventory records, periodic testing is simple to implement. The fifteen-year threshold, which is based upon manufacturers' specifications on average tank life, is a clear criteria providing a basis for when officials need to act. Selecting out those tanks each year, notifying owners, and verifying that tests were done is the kind of procedure local officials are accustomed to conducting successfully.

Secondly, the periodic testing requirement when enforced, creates an incentive for tank owners to voluntarily replace inferior tanks and invest in new, longer term reliable tank facilities. Since 1978, we have not observed a significant number of operations going out of business due to the cost of the testing.

Thirdly, the technologies used in performing periodic tank tightness tests are the only method available to perform a spot test with near absolute certainty. It is therefore essential as a definitive check which will ensure that small leaks occurring after fifteen years do not go undetected for long periods of time.

TABLE 4

UNDERGROUND FUEL & CHEMICAL STORAGE TANKS
LOCAL PROVISIONS FOR SET BACK FROM
PUBLIC SUPPLY WELLS

Town	Set Back
Barnstable	2,000 feet
Bourne	500 feet - fiberglass construction
Brewster	Prohibited within 4 feet of water table and 100 feet of surface water body.
Chatham	1,000 feet
Dennis	700 feet
Eastham	1,000 feet
Falmouth	--
Harwich	1,000 feet - underground storage discouraged in Water Resource Areas
Mashpee	Prohibited within 4 feet of water table and 100 feet of surface water body.
Orleans	500 feet
Provincetown	1,000 feet
Sandwich	Prohibited in P.S.W. Areas
Truro	None
Wellfleet	2,000 feet
Yarmouth	--

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All new tanks being placed in the ground on Cape Cod meet the design standards for new tanks which are presented in the regulations. (Twelve of the fifteen towns require tanks to be set back a minimum distance from water resources.) Table 4 lists these requirements and others such as the prohibition in the Town of Sandwich of new tank installations in the entire public water supply area. Harwich discourages tank installation in water resource districts by imposing burdensome permitting procedures for tanks installed underground in these areas. The Town of Brewster requires board of health review of proposed tank installations in water resource protection districts.

Discussions with local health and fire department officials determined that up-to-date design and installation standards for new tanks are being rigorously enforced. All fifteen towns now require modern tank facilities (tank and piping) which are corrosion resistant. In addition to tank facility design and material standards, towns have also successfully implemented standards controlling tank facility siting for the purpose of protecting ground and surfact water resources.

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Cape Cod Planning and Economic Development Commission

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Model Health Regulation to Prevent Leaking of Underground Fuel and Chemical Storage Systems **(Revised 2/82)**

Under Chapter 111, Section 31 of the Massachusetts General Laws, the ________________ board of health hereby adopts the following regulations to protect the ground and surface water from contamination with liquid fuel or toxic materials from leaking storage tanks. The following regulations apply to all underground fuel and chemical storage systems of 1,000 gallons or greater.

Section 1. Definitions

Abnormal Loss or Gain shall mean a loss or apparent gain in product exceeding 0.5 percent of the volume of product used or sold.

Operator shall mean the lessee or person(s) in control of and having responsibility for the daily operation of the facility for the storage and dispensing of flammable and combustible liquids.

Owner shall mean the person(s) who owns, as real property, the tank storage system used for the storage and dispensing of flammable and combustible liquids.

Section 2. Tank Regulation

2-1. Every owner of an underground gasoline, fuel or chemical storage system shall file with the board of health, the size, type, age and location of each tank, and the type of material stored, on or before ________________. Evidence of date of purchase and installation, including fire department permit shall be included.

2-2. Owners of tanks for which evidence of installation date is not available shall at the order of the board of health have such tanks tested or uncovered for inspection. If in the opinion of the agent of the board of health or head of the fire department, the tank is not product tight, it shall be removed.

Section 3. Inventory Control

3-1. Every underground storage system shall have a method of accurately gauging the volume contained in the tank and a method of accurately metering the quantity of product removed during service. The metering device shall be maintained in accurate calibration. Storage systems in service at the time of passage of this regulation shall be in compliance within 90 days of the effective date of this regulation.

3-2. Accurate daily inventory records, as required by Massachusetts Fire Prevention Regulations-527 CMR 5.05 (3), shall be based on actual daily measurement and recording of tank product and water levels and the daily recording of actual sales, use and receipts. The inventory records shall include a daily computation of gain or loss. The mere recording of pump meter readings and product delivery receipts shall not constitute adequate inventory records.

3-3. The owner and operator shall participate in a program of regularly scheduled inventory verification. Frequency of inventory verification shall be as follows: annually, for systems from which less than 25,000 gallons/month of product is sold or used; semi-annually for systems from which 25,000–100,000 gallons/month of product is used or sold; quarterly for systems from which more than 100,000 gallons/month is used or sold. Owners shall submit annually to the board of health a certified statement that inventory records have been maintained and reconciled as required by Massachusetts fire prevention regulations. Such records shall be made available to the board of health upon their request.
Where the storage tanks are owned by the operator, inventory verification shall be performed by a certified auditor or other independent qualified person approved by the board of health.

3-4. All steel tanks shall be subject to a Petro-Tite (Kent-Moore) Pressure Test or any other pressure test providing equivalent safety and effectiveness 15 years after installation and annually thereafter. The 5 psi air pressure test is a recognized method of leak detection but is not recommended since it can cause explosions.

3-5. Nonconforming steel tanks installed prior to January 1, 1960, shall be removed and properly disposed of by ________________.
All other nonconforming steel tanks installed prior to the effective date of this regulation shall be removed when 20 years old. At such time the exhumed tank shall be examined for leaks. If a leak exists an investigation of amount and location of spilled substance shall be undertaken at the expense of the owner. If, in the opinion of the agent of the board of health, the spilled substance poses a significant threat to health and safety it shall be removed by the owner.

Section 4. Report of Leaks or Spills

4-1. Any person who is aware of a spill or abnormal loss of product shall report such spill or loss immediately to the head of the fire department, and within two hours to the board of health.

4-2. All leaking tanks *must be emptied within 24 hours of leak detection* and either removed or repaired within a time specified by the board of health, under the direction of the head of the fire department.

4-3. Service companies shall report to tank owners and the board of health any unaccounted for significant increase in heating fuel consumption which may indicate a leak.

4-4. All tank installations within four feet of high water table or within 100 feet of a surface-water body shall be of fiberglass construction.

Section 5. Tank Selection and Installation

5-1. All tanks shall be properly installed as per Massachusetts Fire Prevention Regulations and manufacturers specification, under the direction of the head of the fire department. Tanks shall be of approved design and protected from internal and external corrosion. The following tank construction systems are considered to provide adequate corrosion protection: all fiberglass construction; steel with bonded fiberglass or enamel coating and non-corrosive internal lining, and the Steel Tank Institute 3-Way Protective System. All underground storage of chemicals other than gasoline and fuel shall be contained in tanks approved by the agent of the board of health. Any other system must be shown to provide equivalent protection.

5-2. Tanks shall be installed by a manufacturer's approved installation contractor.

5-3. If it is necessary to replace or interior-coat an underground steel tank which developed a corrosion induced leak, all other steel tanks at the facility which are the same age or older whether or not they are leaking shall be interior-coated or replaced with tanks that meet the requirements of 5-1.

5-4. If a cathodic protection system is installed, an ongoing monitoring and maintenance program shall be conducted. If sacrificial anodes have been installed, their proper operation shall be confirmed by a qualified person at least once a year.

5-5. The operator shall notify the head of the fire department prior to the commencement of tank installation. The head of the fire department or the board of health may require repair of protective coatings prior to installation or final cover.

Section 6. Product Storage at Residential Sites

6-1. Following the effective date of this regulation the installation of underground fuel, gasoline or other chemical storage tanks on single-family or two-family residential sites is prohibited.

6-2. All fuel, gasoline or other chemical tanks in service at single-family or two-family residential sites on the effective date of this regulation shall be removed from service 30 years after the date of installation. If the date of installation is unknown, it shall be assumed to be January 1, 1960.

Section 7. Proximity to Water Supplies

7-1. The installation of subsurface fuel, gasoline or other chemical storage systems within 2,000 feet of a public water-supply well is prohibited.

7-2. The board of health may require the installation of one or more ground-water observation wells at any site where fuel, gasoline or other chemical is stored underground within 2,000 feet of a public or private water-supply well. Water samples from such observation wells may be required by the board of health at any reasonable time, and shall be analyzed at the expense of the owner at the order of the board of health.

Section 8. Costs

8-1. In every case the operator shall assume responsibility for costs incurred necessary to comply with this regulation.

Section 9. Variances

9-1. Variances from this regulation may be granted by the board of health after a hearing at which the applicant establishes that the installation or use of an underground storage tank will not adversely affect public or private water resources.

9-2. In granting a variance, the board will take into consideration the direction of the ground-water flow, soil conditions, depth to ground water, size, shape and slope of the lot, existing and known future water supplies.

ELECTRICAL LEAK DETECTION SYSTEMS FOR UNDERGROUND STORED CHEMICALS AND FUELS

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Grand Ledge, MI Washington, DC

Recent US Senate testimony indicates that between 75,000 to 100,000 underground storage tanks are leaking across the country. Growing nationwide interest is aimed at managing the "L.U.S.T." or Leaking Underground Storage Tank problem. The physical environment and chemical characteristics of stored materials must be evaluated so that suitable leak detection and prevention methods and equipment can be designed, selected and installed. In response to some of these needs, technology has been developed to monitor a wide range of stored product, including gasoline, oil, solvents, chemicals, pesticides and hazardous wastes.

In October of 1983, the USEPA launched a two-year research effort aimed at describing the scope and severity of the storage tank problem. A portion of the research effort will be used this fiscal year in a survey of storage tanks including their numbers, age, type of facility, technological devices for leak detection, leak detection methods, reliability, and tank testing for integrity.

At the state level, environmental officials recently warned that the United States is on the brink of a dangerous ten to fifteen year period of gasoline leaks from underground storage tanks. Because of the environmental threat posed by leaking underground tanks, and the need to begin upgrading facilities, several states now have standards for tank permitting. At the time of this writing, six states have already passed legislation for permitting and monitoring underground hazardous material tanks. Thirty-two other states propose similar legislative action.

Costs for environmental impairment liability insurance are rising rapidly due to the expense and complexity of contamination clean-up projects. Insurance companies are interested in a method of reducing their risk as more and more case histories show the potential liability of seemingly insignificant underground storage tank leak problems. Thus, early detection makes it possible to limit product loss from tank leakage and may assist in negotiating reductions of insurance premiums.

An erroneous assumption is that all underground and aboveground fuel and chemical storage tanks develop leaks according to a time table. Some tanks develop leaks as early as their second year, and sometimes immediately, due to damage during installation. Others have been known to leak as late as their 48th year. The average leak reportedly occurs at approximately 18 years (National Oil Jobbers Council, 1982). Many factors influence these inevitable leaks -- including bacteria, soil, and electrical current corrosion, improper installation, and tank and piping failure.

Leak detection systems can be applied to situations where chemicals or fuels are stored either above or underground. Underground storage tanks are invisible, out of sight, forgotten, and often neglected. Therefore, many serious liability problems are associated with leaks from underground storage tanks. There are several important reasons why leak detection systems have been developed and are considered as an important ground water monitoring method to manage underground storage tanks.

Product loss is one of the key reasons. Several thousand gallons of gasoline is a considerable investment. Even with losses of one to two drops per second, or approximately 5/100 of a gallon per hour, the yearly loss is nearly 450 gallons. At $1.20 per gallon, that's over $550 per year, and that is for a leak at or below the generally accepted standard as established by the National Fire Protection Association for Underground Tank Tightness.

Second, consider the problem of widespread contamination. A 450 gallon gasoline leak has the potential of contaminating a vast amount of soil and ground water (Fronczak, R.E. and Hersey, K.K., 1982). The costs to remove the tank and excavate contaminated soil can make that $550 product loss figure pale in comparison. Industry sources estimate that a simple tank clean-up can cost $70,000; if the soil has to be cleaned up and the tanks removed, costs can soar up to $1,000,000. If liquid, especially gasoline, gets into surface water, sewers or utility trench, clean-up costs could be well above this (Shaner, J.R., 1982). State conservation or environmental agencies commonly levy a $10 to $15 per fish fine when contamination results in major fish kills. Soluble constituents in various products can contaminate public drinking water supplies, rendering municipal well supply fields inoperative and contaminating private, residential, or industrial wells (Le Grand, H.E. and Brown, H.S., 1977). Clean-up of serious ground water contamination from leaks can be extraordinarily costly, and time consuming, and may not be effective in fully restoring aquifer quality.

Another reason leak detection is being considered is to reduce the risk of other liabilities, including those from insurance claims. A major oil company had to pay between $10,000,000 to $12,000,000 because of a gasoline leak which infiltrated a sewer system, and forced purchase of a row of 25 homes in suburban Denver. Buying and maintaining insurance against these super-liabilities is becoming more difficult and costly.

Hazardous materials leaking from tanks also expose workers and surrounding communities to dangers. Personal injury, property damage, and third party liability can occur when tanks leak. One company found this out when they paid $8,000 per adult and $3,000 per child to 21 families in a 1978 gasoline leak in New York. Early warning systems can reduce the possibility of personal injury and property damage.

Finally, leak detection systems are beneficial because they help industry comply with state and federal regulations for protection of groundwater quality. In California, new legislation calls for a statewide local government program for permitting and monitoring underground hazardous material tanks and reporting leaks. The State Department of Health Services is required to develop regulations for the control of such tanks. Owners of tanks installed before January 1, 1984, are required to install a monitoring system capable of detecting unauthorized releases. Other states and even local governments are developing similar laws concerning storage tank registration and regulation. Public attention through the media is causing development of such regulations, which will likely apply to most businesses with tank storage.

Several types of leak detection systems currently are available. They can be divided into three groups:

- Manually assisted
- Continuous, in-situ
- Sample drawing

The manually assisted electrical leak detection systems are usually light weight, portable, self-contained units that provide accurate measurements of the presence of, depth to, and thickness of lighter than water immiscible liquids, such as gasoline, petroleum, etc. These units also measure depth to the Immiscible Lighter Than Water (ILW)/water interface level, depth to the water table, and sometimes also may be able to detect heavier than water immiscible liquids. These units are usually powered by self-contained, low voltage DC batteries, and may be used within observation wells as small as 2 inches in diameter.

The second type of electrical leak detection unit is a permanent in-situ probe or cable which monitors the presence of specific products. Various monitoring techniques are used such as:

- Electrical capacitance
- Electrical conductivity
- Electrical conductivity/pressure, temperature
- Ionic concentration
- Conductivity, refractivity
- Thermal conductivity
- Coaxial, semipermeable membrane

These units monitor immiscibles, either lighter or heavier than water, or miscibles. One of the physical techniques used to monitor immiscibles lighter than water is the use of a floating circular probe. This probe floats on top of the water within a ground water monitoring well. The probe may vary in diameter and can monitor various products based on a number of factors at each specific application, such as: type of product, diameter of the ground water monitoring well, and tank or tank farm locations. Probes submerged in water are also used to detect heavier than water product. Drop cables are available for trench application to monitor piping leaks, and use either a degradable or semi-permeable coaxial membrane.

The monitoring of miscible product leakage is more difficult due to the dispersion and dilution tendency of the product. In some applications, floating circular probes have been successful in monitoring miscible product with careful well design and placement.

A sample drawing analyzer is commonly used to measure changes in the Total Organic Carbon content in the ground water sample. This type of unit is located in the proximity of the tanks, and draws ground water samples, automatically tests them for Total Organic Carbon, and records the test analysis result.

Technology will continue to provide an array of electrical monitoring systems which can help industry to manage underground storage tanks. Nevertheless, the initial site investigation and facility evaluation supply much of the information necessary to design and install such a system successfully. It would be very convenient to suggest one system which would monitor all products in all situations, but this is not the case. Experience has shown that industrial facilities have site-specific characteristics which must be addressed in the selection of the most appropriate detection system, including:

1. Facility design data
    a) Plant layout with respect to individual tanks or tank farms
    b) Tank specifications (age, as-built, location, volume, etc.)
    c) Product characteristics and storage records, and product use records
    d) Utility layout, underground piping between tanks and buildings
    e) Tank filling procedures
    f) Past records of spills or leaks

2. Available regional information:
    a) Adjacent property usage: potential liability exposure
    b) Location of municipal, private, and plant wells
    c) Boring logs, and other geotechnical data from previous on-site construction activities
    d) Soils information
    e) Bedrock geology
    f) Hydrogeologic characteristics
    g) Ground water quality in the vicinity of facility

Prevention of industrial groundwater pollution can be a difficult and complex undertaking at many facilities. However, management of existing and proposed underground storage tank facilties may be simplified by appropriate use of an electrical leak detection system. A priority ranking for tanks or tank farms can be established based on review of the information outlined above, plus other data that may pertain to a specific facility. Once this is done, a program plan for leak detection can be developed. A key part of the plan will be the design of the detection system and the selection of appropriate devices. This must be followed by practicing sound installation procedures for placement and hook-up of the detection system. Once the system is functioning in-place, its continuing value in early detection of leaks must be safeguarded by careful attention and an on-going program of periodic monitoring and maintenance.

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2. LeGrand, H.E. and Brown, H.S.. 1977. "Evaluation of Ground Water Contamination Potential from Waste Disposal Sources", USEPA Contract Number 68-01-4405. 1977.

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4. Shaner, J.R.. July 1982. "How Nondetection Can Cost You - The Tank Leak Mess." National Petroleum News.

ABSTRACT

MANAGEMENT OF GASOLINE LEAKS - A POSITIVE OUTLOOK

by Melvin L. Blevins* and
Dennis E. Williams**

Gasoline containment and transmission facilities constructed in close proximity to fresh groundwater reservoirs poses increasing risks of contamination by leakage from these facilities. The magnitude of this risk can be seen by the tens of thousands of old steel storage tanks and hundreds of miles of buried pipelines overlying potable groundwater supplies. In addition, increasing construction of new facilities is taking place, adding more risk to an already serious problem.

Regulatory agencies have recognized this problem and several promising monitoring and control programs are presently being developed. In the past 15 years, a sufficient amount of data has been collected to propose an effective method to restore the gasoline contaminated aquifer to near its original quality. This method involves three distinct phases: (1) detection and containment; (2) free gasoline recovery; and (3) bacterial degradation.

It is important that early discovery and containment of any suspected leak be made as soon as possible so that recovery procedures may be started and the area of gasoline removal minimized. The first phase of the clean-up process involves removal of free gasoline to the point where no more drains by gravity resulting in only pellicular gasoline adhering as films to the surfaces of the porous media. Once recovery of all free gasoline is complete, areation of the contaminated zone is accomplished, (when geohydrologic conditions permit) by lowering the water table below the zone of suspected contamination. This areation promotes rapid bacteriological degradation of the remaining pellicular gasoline be several bacterial species (Pseudomonas and Arthrobacter) which utilize gasoline as a source of energy.

Results from a major gasoline leak in the Forest Lawn area of Glendale, California in 1968 supports the effectiveness of this technique. Examples on the use of this clean-up procedure area applied to several current gasoline leaks in the Southern California area.

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MANAGEMENT OF GASOLINE LEAKS - A POSITIVE OUTLOOK

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Melvin L. Blevins* and
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INTRODUCTION

California uses over five million acre-feet of water each year for domestic water purposes. Approximately half of this water comes from 400 groundwater reservoirs located from 20 to 1,000 feet below the ground (DWR 1975). Pollution of any of these reservoirs therefore is a serious problem to the health and welfare of the citizen of California.

It is estimated that there are several hundred thousand of underground tanks throughout the state which are used to store hazardous chemicals, fuels, and waste products. In addition, hundreds of miles of buried pipelines transport petroleum products over productive groundwater reservoirs.

At present there is no automatic warning system with which to alert the public as to contamination or potential contamination of groundwater from these tanks and pipelines. Generally, the first warning comes as the result of adverse health effects noticed in domestic or municipal supplies. By that time however, the contamination has usually degraded the aquifer to the extent that the original quality may never be restored. It is important therefore to recognize potential hazards to groundwater supplies and to effect an efficient control program to ensure their protection.

Recent legislation has been passed by the State of California which is attempting to identify and regulate the usage of underground tanks for storage of hazardous chemicals and fuels. However, the monumental task of identifying problem sites, detecting leaks, and effecting clean-up will take years to enact.

The purpose of this paper is to focus on one aspect of groundwater contamination, that of leakage of gasoline into productive aquifers. This paper, in part, is a follow-up of results from a major gasoline spill which occurred over 15 years ago in the Forest Lawn area in Glendale, California. Based on results from that study in addition to other work, a positive outlook for clean-up of gasoline contaminated aquifers is presented.

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BACKGROUND OF GASOLINE LEAKS INTO GROUNDWATER RESERVOIR

It is estimated that there are tens of thousands of steel gasoline storage tanks buried at filling stations in the United States and hundreds of miles of underground pipelines which transport petroleum products (McKee, 1972). Until the Forest Lawn, Glendale leak of 1968 no reported gasoline leakage had endangered a major groundwater reservoir in the United States.

Undoubtedly, the probability of undetected gasoline leaks is high. This may be reasoned from the large numbers of buried tanks and pipelines and corrosion and time factors alone. Undetected leaks most probably occur in areas where groundwater exploitation is minimal or in hydrogeologic environments which prevent contamination from overlying leaks (e.g. confined aquifers). The question then arises as to the rate and extent of gasoline movement as well as the eventual state of the contaminated zones.

GASOLINE MOVEMENT IN POROUS MEDIA

Movement of gasoline through porous media is controlled by both gravitational and capillary forces. Since leaks usually originate in the unsaturated zone, the original movement is primarily gravitational resulting in a vertical flow field. Initially, the downward moving gasoline occupies a percentage of the interstices known as the "residual saturation" (Buono, 1982). For gasoline this residual saturation is about 10% of the total porosity (Dietz, 1971) (see Figures 1 and 1A).

Continued vertical migration of gasoline requires saturation in excess of the residual saturation. With large leaks, a sufficient supply of gasoline is provided so that the residual saturation is met in the entire vertical column of unsaturated material from the leak site to the water table. Once this occurs, a layer of "free gasoline" begins to build up on the water table and the flow direction turns from vertical to horizontal (Figure 2).

The relationship of the mound of free gasoline above and below the water table governed by :

$$d = h \frac{(\gamma_w - \gamma_g)}{\gamma_g}$$

Where:

d = depth of free gasoline below the average plane of the water table (ft).
h = height of free gasoline mound above the water table (ft); (h = 2.3 d).
γ_w = Specific weight of fresh water (62.4 lbs/ft)
γ_g = Specific weight of gasoline (43.7 lbs/ft)

Equation 1 shows that for every foot of free gasoline floating on top of the water table, a "pocket" of free gasoline

FIGURE 1

VERTICAL MOVEMENT OF GASOLINE
ON FLUCTUATING WATER TABLE

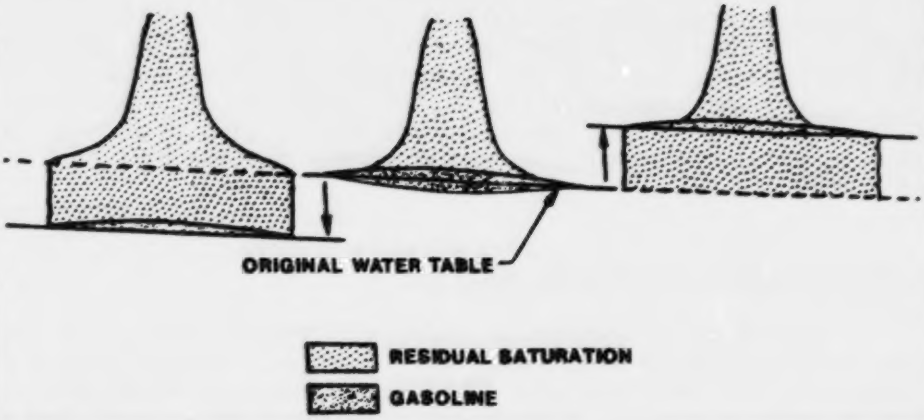
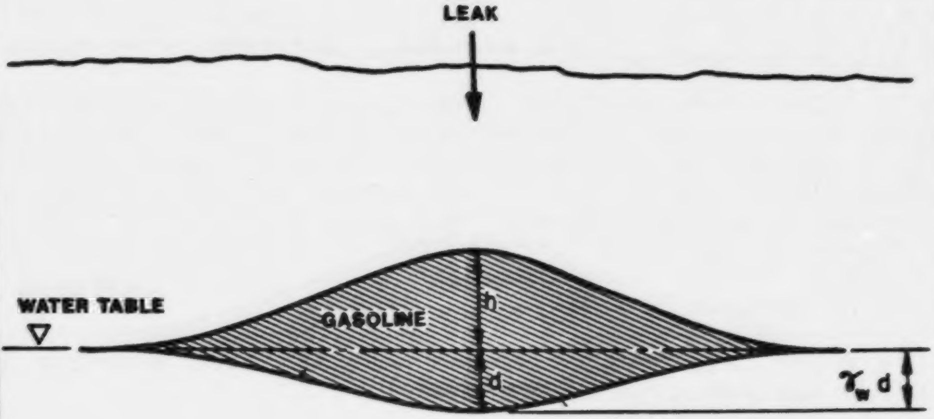

ORIGINAL WATER TABLE
RESIDUAL SATURATION
GASOLINE

FIGURE 1A


LEAK
WATER TABLE
GASOLINE
h
d
$\gamma_w d$

$$\gamma_w d = (h+d)\gamma_g$$

WATER TABLE - GASOLINE INTERFACE

VERTICAL AND HORIZONTAL MOVEMENT OF GASOLINE

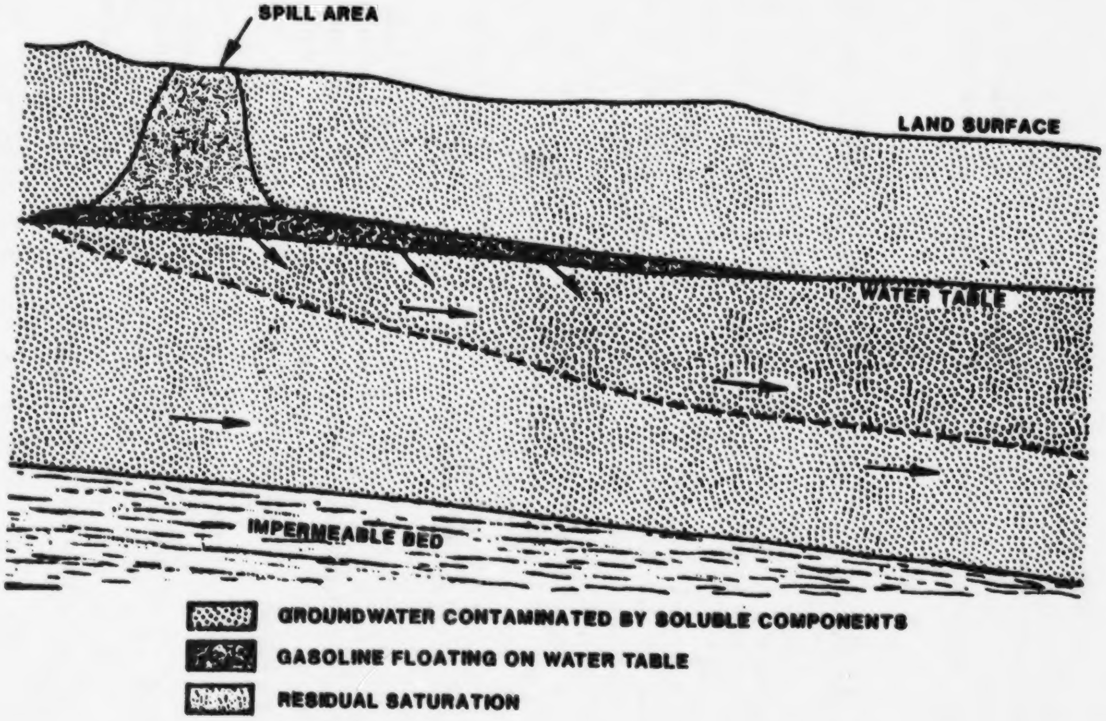

SPILL AREA
LAND SURFACE
WATER TABLE
IMPERMEABLE BED
GROUNDWATER CONTAMINATED BY SOLUBLE COMPONENTS
GASOLINE FLOATING ON WATER TABLE
RESIDUAL SATURATION

FIGURE 2

is depressed 0.43 ft below the water table (see Figure 1A). Free gasoline flows on top of the water table under the same hydraulic gradient as the aquifer water but under a hydraulic conductivity 1.67 greater than that of water (due to viscosity differences). Movement of gasoline through a porous medium will continue both vertically and horizontally until the supply is interrupted and the degree of saturation decreases below the residual level.

In addition to contamination of the immediate zone of flow overlying the water table, the slightly soluble components (e.g. benzene and toluene) may cause a degradation of the aquifer in general. Also, the volatile nature of the vertical migration plume may cause diffusion of gasoline vapors in the unsaturated zone.

CURRENT REGULATION REGARDING GASOLINE LEAKS IN GROUNDWATER

In California, the new requirements for underground storage tanks and sumps used to store hazardous substances are found in provisions of Assembly Bill 1362 (Sher Bill) and Assembly Bill 2013 (Cortese Bill) approved by the Governor on September 23, 1983, and also in a new underground tank program currently being implemented by the California Regional Water Quality Control Board, Los Angeles Region. In order to implement the provisions of the Assembly Bills, it will be necessary for the State Water Resources Control Board, the Regional Water Quality Control Boards, the State Department of Health Services, and local (city or county) permitting agencies to develop specific policies and regulations. It is anticipated that these will be completed early in 1984.

Assembly Bill 1362 basically establishes a statewide local government program for permitting and monitoring underground tanks and sumps used for the storage of hazardous substances and for the reporting of leaks. Tank and sump owners have the following reponsibilities under this Assembly Bill:

(1) Permits to operate must be obtained from local agencies (city or county) for all new and existing underground storage tanks and sumps used to store hazardous substances. A permit fee shall be paid for each tank or sump and the permit shall be effective for 5 years.

(2) Monitoring systems, or approved monitoring system alternatives, must be installed by January 1, 1985 on all tanks and sumps which were installed before January 1, 1984.

(3) Underground storage tanks installed after January 1, 1984 are to be designed and constructed to provide primary and secondary levels of containment and a monitoring system which is

capable of detecting the presence of hazardous substances between the primary and secondary containments.

(4) Annual reports must be submitted to local permitting agencies which detail any changes in the tank usage, storage of new hazardous substances, changes in monitoring procedures and unauthorized releases.

(5) Any unauthorized release or leak from an underground storage tank is to be reported to the local permitting agency within 24 hours after the release and a full written report is to be submitted within 5 working days after the release.

(6) No underground storage tank or sump shall be abandoned until specified procedures are followed and it is demonstrated to local permitting agencies that it is safe to do so.

(7) Civil penalties ranging from $500 to $5,000 per day may be assessed against tank operators who fail to comply with the above requirements.

Assembly Bill 2013 requires the State Water Resources Control Board to establish a statewide inventory of all concrete sumps, nonvaulted buried tanks or other underground containers used to store hazardous substances. Owners of such tanks and sumps are required to file a "Hazardous Substance Storage Statement" and pay a $10 fee by July 1, 1984 for each tank or sump. Owners or operators who fail to submit such statements and fees by January 1, 1985 will be liable for civil penalties ranging from $500 to $5,000 per day for each day the statement has not been received.

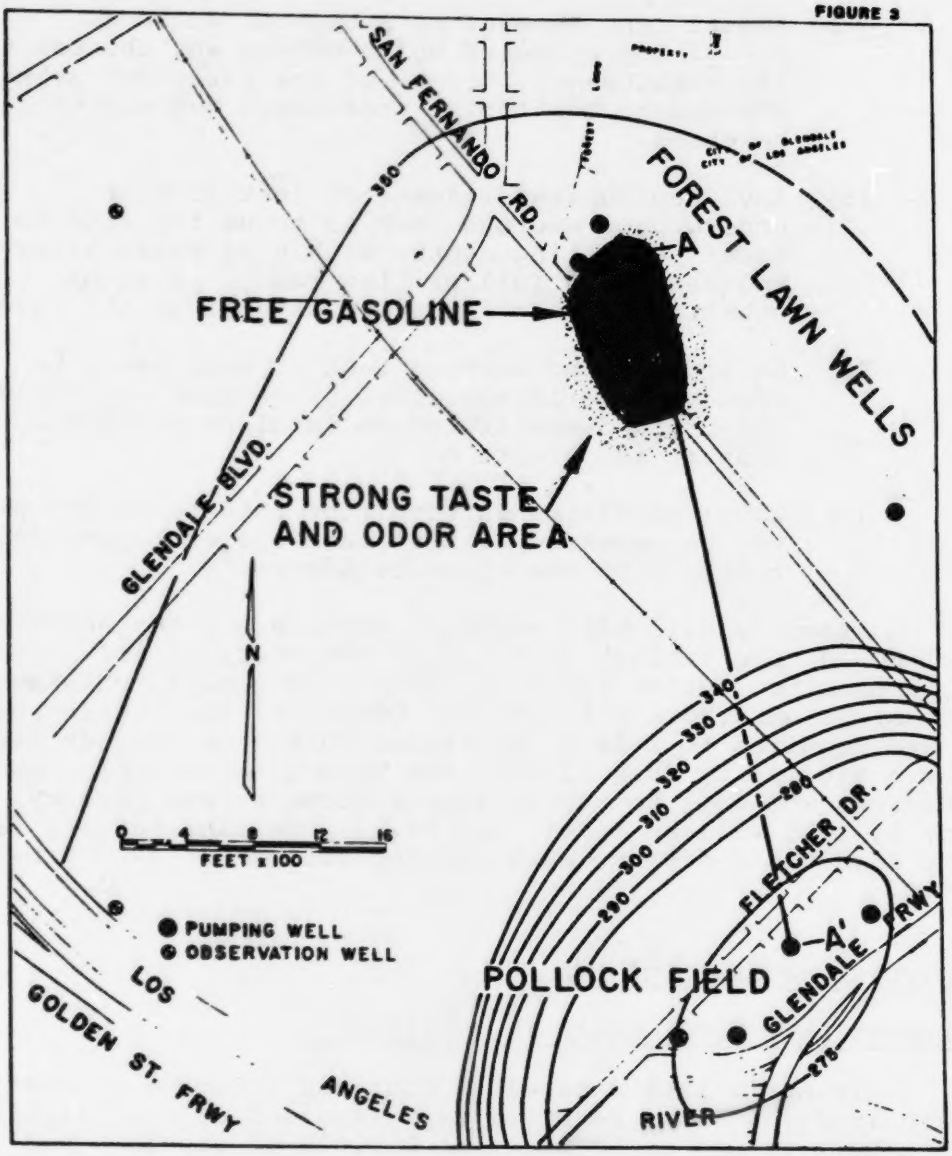
CASE STUDIES OF GASOLINE LEAKS

Forest Lawn Area, Glendale, California

In April 1968 a gasoline odor was detected at a drinking fountain in Forest Lawn Memorial Park in Glendale, California. By September, it was estimated that 250,000 of gasoline had leaked from a buried 8-inch pipeline into the groundwater reservoir in the area. The contamination first surfaced in the Forest Lawn irrigation well (FL #4).

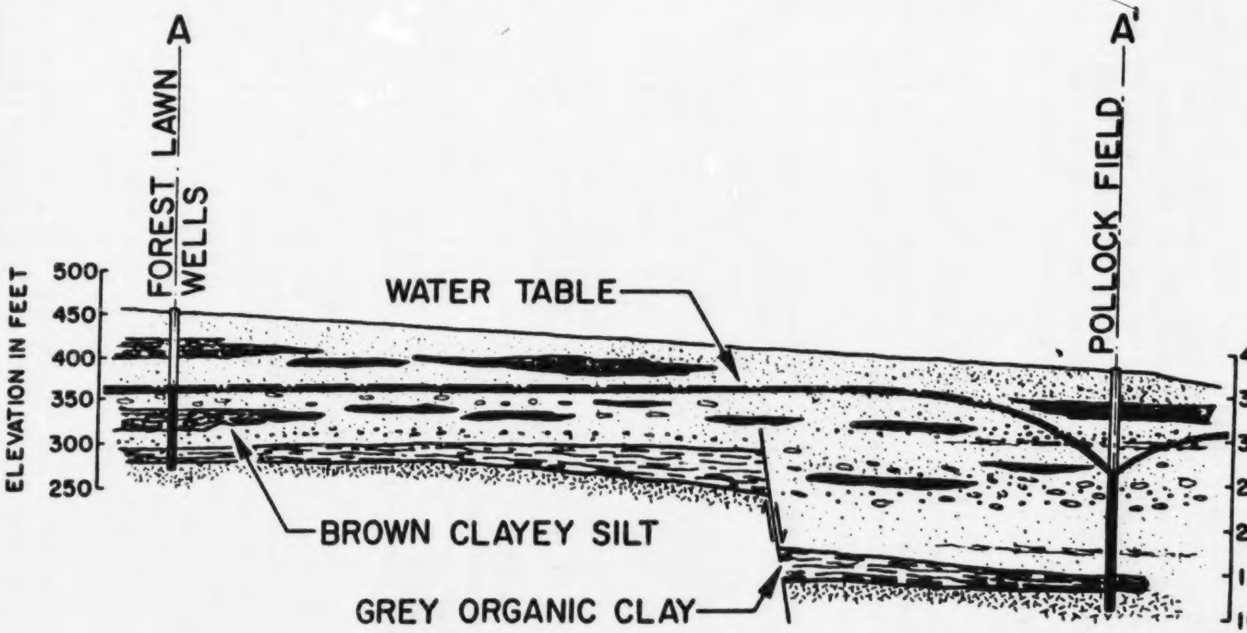
The problem was extremely critical as the leak site lay between two major well fields supplying water to the City of Los Angeles (see Figures 3 and 4). Both well fields had created significant cones of depressions and it was only the fact that the leak site was on the groundwater divided between the two cones which prevented migration into either of the well fields.

FIGURE 3


PROPERTY
CITY OF GLENDALE
CITY OF LOS ANGELES
SAN FERNANDO RD.
FOREST LAWN WELLS
350
A
FREE GASOLINE
GLENDALE BLVD.
STRONG TASTE AND ODOR AREA
N
0
4
8
12
16
FEET x 100
● PUMPING WELL
◉ OBSERVATION WELL
340
330
320
310
300
290
290
280
FLETCHER DR.
FRWY
A'
GLENDALE
POLLOCK FIELD
275
LOS
ANGELES
RIVER
GOLDEN ST. FRWY

CONTAMINATED AREA AND GROUNDWATER CONTOURS (1968)

FIGURE 4


A
FOREST LAWN WELLS
A'
POLLOCK FIELD
ELEVATION IN FEET
500
450
400
350
300
250
WATER TABLE
BROWN CLAYEY SILT
GREY ORGANIC CLAY
400
350
300
250
200
150
100

GEOLOGIC CROSS SECTION - FOREST LAWN AREA

Immediate action was taken to find the extent of the leak and by August, 1969, some 30 wells had been drilled. During the course of the drilling, free gasoline was found on the water table ranging from 12 to 30 inches and it was estimated that free gasoline ranging from 6 to 18 inches covered approximately 160,000 square feet of surface area (Williams, 1971).

Skimming wells were installed and by October, 1969, approximately 20,000 gallons of free gasoline had been recovered. By November, 1969 however, free gasoline appeared in a well 500 feet closer to one of the major supply well fields (Pollock field). The Los Angeles Department of Water and Power (LADWP) immediately stopped all pumping from those wells in an effort to reverse the hydraulic gradient. These wells were not turned back on until May, 1973.

Further drilling and testing by both the LADWP and the Western Oil and Gas Association (WOGA) resulted in containment and removal of free gasoline by creation of localized cones of depression confining the gasoline to the area where skimming wells existed (See Figure 5).

From September 1968, through August 1971, about 50,000 gallons of free gasoline floating on top of the water table had been recovered mainly by the skimming process. By mid-1971, the amount of free gasoline was limited to a few wells in the immediate vicinity of Forest Lawn with depth of free gasoline being less than one-half inch (API, 1972).

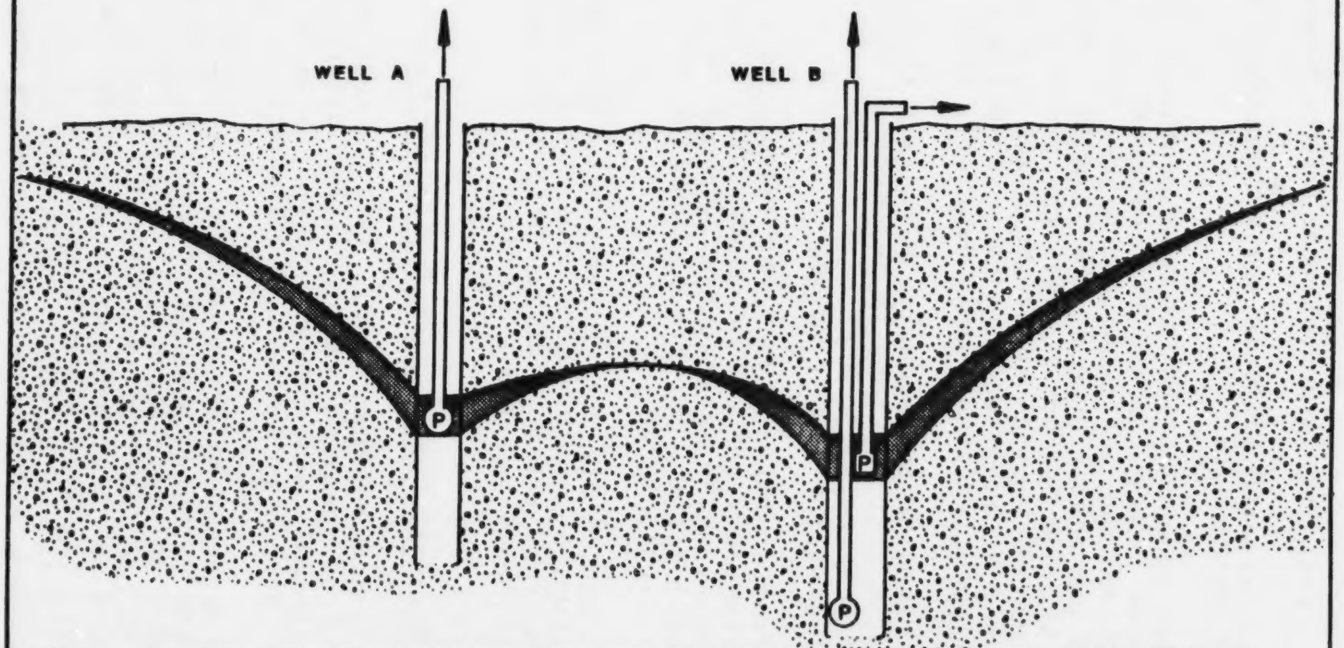
By August 1971, removal of free gasoline by skimming wells was essentially complete and the program was reduced to that of monitoring. Strong taste and odor, however, still occurred in most of the wells in the Forest Lawn area and the Pollock well field and was therefor not useable for supply. The taste and odor were caused from residual saturation of the aquifer materials by "pellicular" gasoline coating the grains which was not released by gravity drainage and skimmed off.

At that time, studies by WOGA consultant McKee (1972) showed that the pellicular gasoline would be metabolized by several species of naturally occurring bacteria (Pseudomonas and Arthrobacter). A year of sampling through August, 1971 showed concentrations of gasoline degrading bacteria of 50,000 per ml or higher in wells which contained traces of free gasoline (McKee, 1972). Concentrations of 5,000 to 50,000 were found in wells which still had gasoline taste and odor while only 200 per ml were found in wells showing no trace of gasoline.

From March 1971 to December 1976 a series of progress reports were made to the Los Angeles Regional Water Quality Control Board (LARWQCB) regarding cleanup of groundwater from the Forest Lawn leak. The reports presented findings of the monitoring program by WOGA consisting of:

FIGURE 5

PUMPING DRAWDOWN AND SKIMMING WELL OPERATION



WELL "A" IS MAINTAINING DEPRESSION AND RECOVERING GASOLINE IN SINGLE OPERATION BY DRAWING FLUID SURFACE DOWN TO PUMP LEVEL.

WELL "B" IS MAINTAINING DEPRESSION BY PUMPING CLEAN WATER FROM BELOW WATER TABLE. FLOATING GASOLINE IS RECOVERED BY SUCTION PUMP AT FLUID SURFACE.

1. Observation of any change in color of gasoline-sensitive paste on a weighted tape to measure depth of free gasoline or even a trace of free gasoline.

2. Sniffing to detect odor in the well.

3. Infrared analysis for hydrocarbons in water samples from wells.

After January, 1975, WOGA discontinued bacterial analyses of water samples from wells and the measurement of atmospheric oxygen in well columns and dissolved oxygen below the water surfaces with the concurrence of the LARWQCB. Between May 15, 1976 through October 5, 1976 the presence or absence of gasoline as measured by the sensitive paste on a weighted tape showed no traces of gasoline. In fact, no wells have exhibited any traces of gasoline since early February, 1976.

Sniffing tests conducted bi-weekly showed no odor since April 1976. The sniffing test is believed to be the most sensitive and most reliable analysis to detect the presence of any traces of gasoline (WOGA, 1976).

Infrared (IR) analysis for hydrocarbons performed bi-weekly during July through September 1976 showed no IR readings as high as 2 mg/l which is essentially negative as any IR analysis less than 5 mg/l (in the absence of gasoline odor) is insignificant (WOGA, 1976).

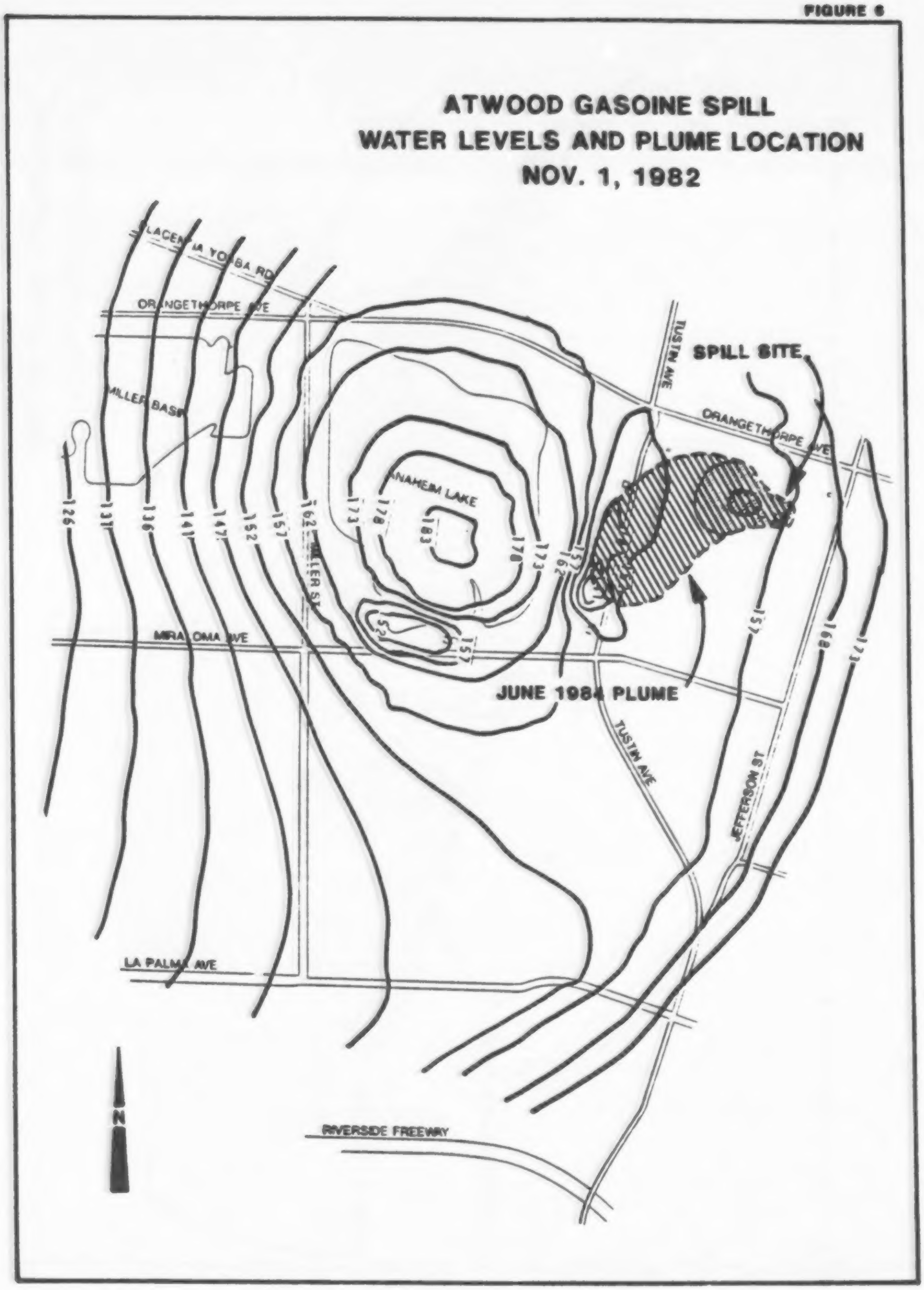
Because of the lack of any detectable gasoline odor or significant level by analysis, on December 31, 1976 the monitoring program at the Forest Lawn site was officially terminated by the Regional Water Quality Control Board.

Atwood Terminal Leak - Anaheim, California

On Monday, September 27, 1982, a leak of approximately 37,000 gallons of gasoline occurred from surface storage tanks at the Atwood Terminal located near the corner of Orangethorpe Avenue and Jefferson Street in Anaheim, California.

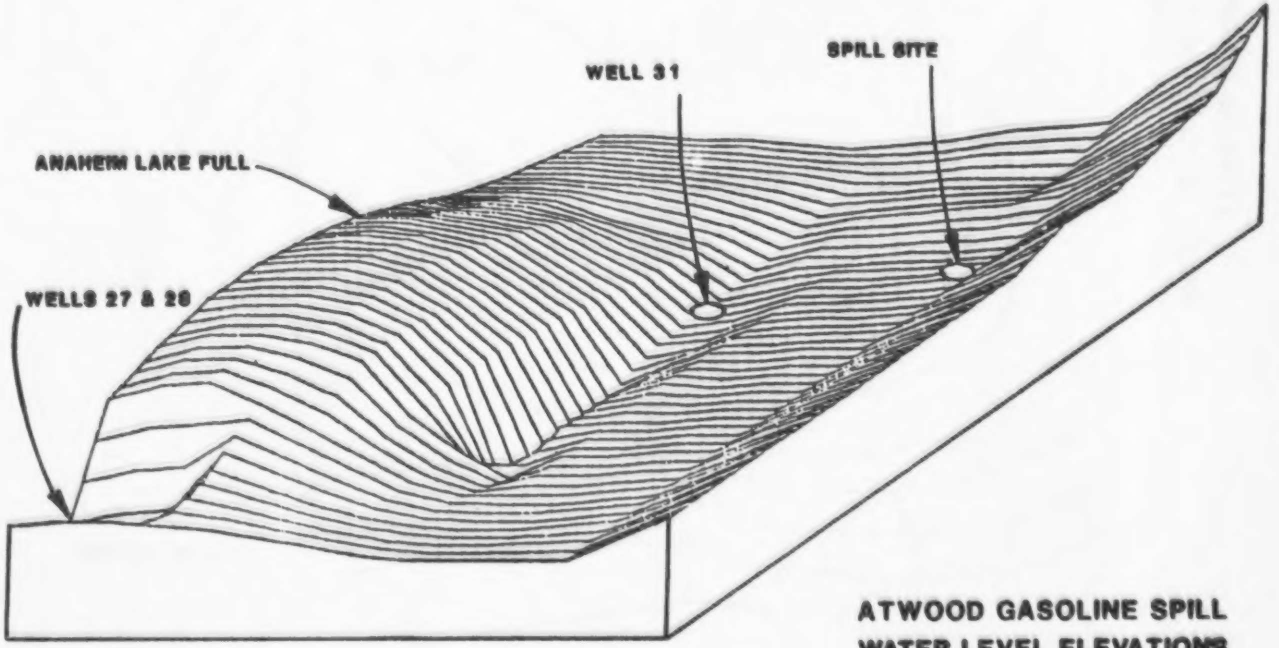
As managers of the groundwater resources within the spill area, the Orange County Water District had an obligation to know the extent and rate of movement of the gasoline plume. Because of the proximity of the spill to several municipal production wells as well as a major aritifical recharge facility (Anaheim Lake), the area was modeled for both flow and solute transport using the existing Orange County Water District groundwater mathematical model. It soon became apparent that the dominant hydraulic force within the leak area was the recharge mound created by Anaheim Lake. During periods when the lake is empty, water levels in nearby wells decline. Conversely, when the lake is full, water levels in the wells rise. Figures 6 and 7 show two and three-dimensional contour plot of water-level elevations showing the effect Anaheim Lake has on the surrounding water levels.

FIGURE 6



ATWOOD GASOINE SPILL
WATER LEVELS AND PLUME LOCATION
NOV. 1, 1982

PLACENTIA YORBA RD
ORANGETHORPE AVE
MILLER BASIN
TUSTIN AVE
SPILL SITE
ORANGETHORPE AVE
ANAHEIM LAKE
126
131
136
141
147
152
157
162
173
178
183
178
173
162
157
152
157
MILLER ST
MIRALOMA AVE
JUNE 1984 PLUME
157
168
173
TUSTIN AVE
JEFFERSON ST
LA PALMA AVE
N
RIVERSIDE FREEWAY


SPILL SITE
WELL 31
ANAHEIM LAKE FULL
WELLS 27 & 28

ATWOOD GASOLINE SPILL
WATER LEVEL ELEVATIONS
(NOV. 1, 1982)

FIGURE 7

Another controlling factor in the flow regime was the pumping of large capacity production wells in the area. Wells No's 27, 28, 30, 31, and 32 created significant cones of depressions in local water levels causing "pumping holes" to form in certain areas. The combination of Anaheim Lake recharge and heavy well pumping is analogous to operation of a seawater barrier in conjunction with a pumping trough, a very effective method of controlling the movement of contaminants.

Model Simulation of the Gasoline Movement

In conjunction with the groundwater flow model, a detailed solute transport model was constructed of the area using an improved version of the Prickett-Lonquist "Random Walk" technique. In this model, movement of particles (e.g. contaminants) is simulated by a convective transport term (as obtained from the flow model) and a random term representing hydrodynamic dispersion.

In modeling the Atwood Terminal gasoline spill, 2,000 particles were initially placed in node (6,25) of the model. These particles may be equated to a volume of free gasoline (i.e. each particle representing 18.5 gallons if the initial spill is 37,000 gallons). However, due to many unknown field processes (e.g. adsorption and the random nature of retardation and subsequent release of gasoline on a microscopic or pore level), modeling of particles in this manner represents a "worst case" situation.

In the model it was assumed that gasoline and water are immiscible and that movement of gasoline occurred on top of the water table with the rate of movement controlled by the prevailing hydraulic gradient. Also, the gasoline moves faster than water under the same driving gradient due to its lower viscosity.

Initially, the movement of the plume was due west towards Anaheim Lake and the pumping depression created by Well No. 31. After reaching this area in mid-January of 1983, the prominent "wall" formed by Anaheim Lake recharge caused the movement to turn southward towards the pumping depression created by Wells 30 and 32. In addition, this movement was accelerated by the cessation of pumping of Well 31. Since February of 1983, the plume has maintained the southerly direction with only a few particles continuing eastward. It seems that a "quasi-steady state" condition has developed over the past few months whereby particles are accumulating in the area between Well 31 on the north and Wells 30 and 32 on the south. The movement is controlled by the present water-level distribution.

The reason that no gasoline has been detected from the discharge of Wells 30 and 32 is due to the well construction. These wells were constructed using the "cable tool" method whereby casing is jacked or driven into the formations and perforated after drilling is complete. There is no artificial filter to allow for easy vertical migration of contaminants in the near-well zone. This fact coupled with deep perforation intervals is believed to have resulted in the gasoline "floating" on top of the water level in the vicinity of these wells, unable to reach the pump intake due to both the deep perforations as well as low vertical hydraulic conductivities.

At present the program is in the first stage of clean-up, (i.e. detection and containment). If model studies prove accurate, the plume is contained between Anaheim Lake on the west and the hydraulic gradients existing the south and east.

Gasoline Tank Leak - LADWP General Office Building

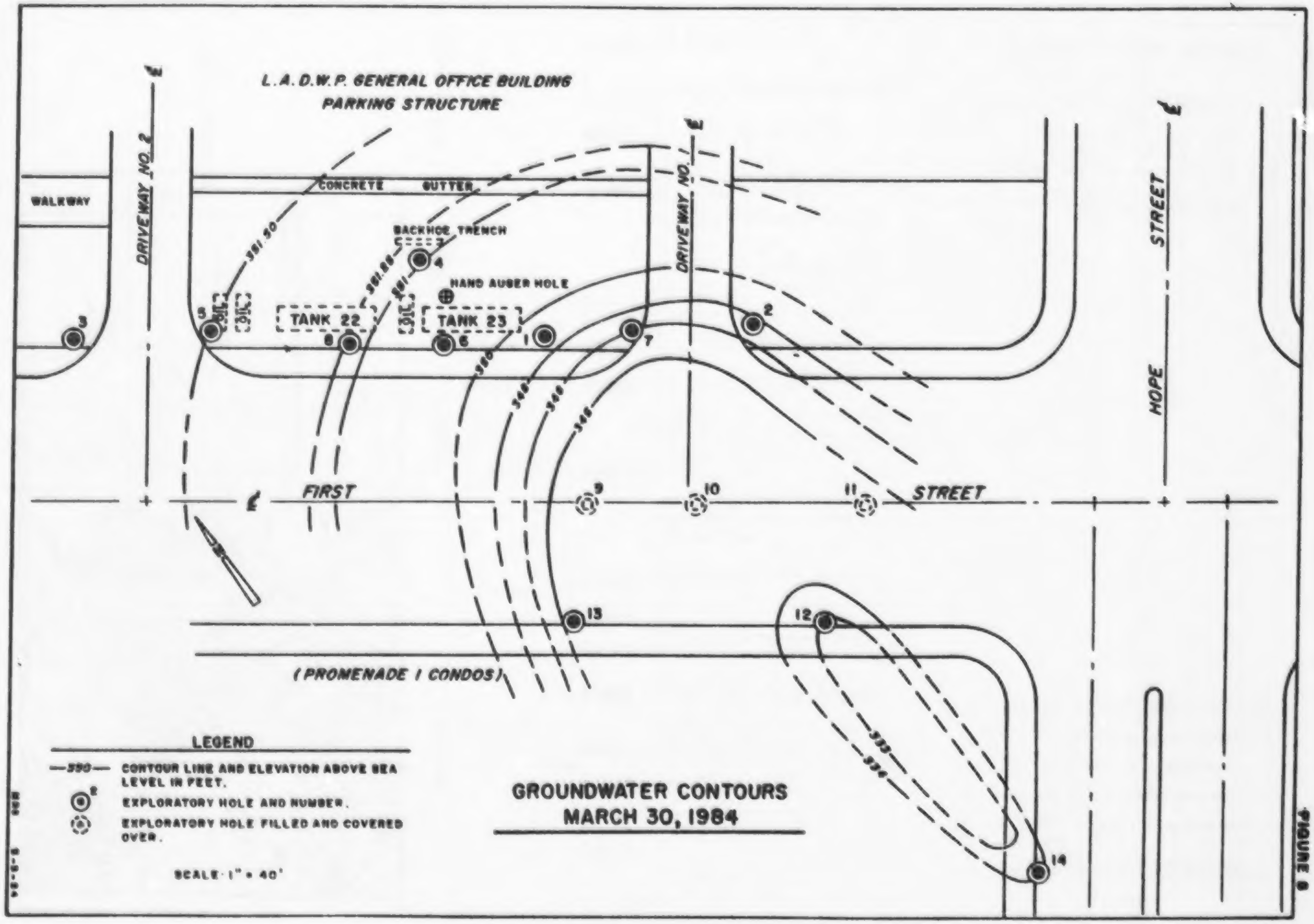
Tank 23 (see Figure 8) of the Los Angeles Department of Water and Power, General Office Building (LADWP/GOB) had been suspected of leaking because of inventory discrepancies. As a result of these discrepancies, a 3-day pressure test (at 5 psi, using nitrogen gas) was conducted starting January 14, 1984. During the test, about 1,350 gallons were discharged from the tank. At the end of the test, the tank had about 25 gallons of gasoline left. Following the pressure test, a hand auger hole was drilled about five feet north of the tank to a depth of 13 feet (Figure 8). A gasoline-saturated formation was encountered. As a result of this preliminary investigation 14 additional boreholes (wells) were drilled to define the horizontal and vertical limits of the gasoline migration.

The geology in the area of the LADWP/GOB was investigated in 1960, prior to the construction of the foundation for the GOB. The geology is described as Miocene Age marine deposits of firm clay shale with interbedded sandstone. The bedrock material is generally very fine grained with low permeability, which has limited the areal extent of the gasoline migration. Overlying the shale are deposits of natural clay soil and varying amounts of fill. Refer to Figures 9 and 10, adapted from a plate of the California Department of Water Resources, and Figures 11 through 14.

A geologic section was prepared by the California Department of Water Resources (1961), which depicts the groundwater hydrology of an area about one mile east of the LADWP General Office Building (Figure 10). The study site is in an area known as the Elysian Hills, with bedrock relatively close to the ground surface. The bedrock, named the Puente Formation, is generally of low permeability comprising alternating strata of siltstone, sandstone, and shale.

The groundwater occurs as percolating water flowing through the higher permeability alluvium and fill materials overlying the bedrock, or in perched zones trapped by the clayey or shaley strata. Groundwater flow for the region is generally in the south or southeast direction from the downtown area of Los Angeles with opportunity for recharge to the alluvial aquifer, called the Gaspur Zone of the Coastal Plain (Figure 10).

FIGURE 8


L. A. D. W. P. GENERAL OFFICE BUILDING
PARKING STRUCTURE
WALKWAY
DRIVEWAY NO. 2
DRIVEWAY NO. 1
CONCRETE GUTTER
BACKHOE TRENCH
HAND AUGER HOLE
TANK 22
TANK 23
HOPE STREET
FIRST STREET
(PROMENADE I CONDOS)
351.50
351.85
351
350
348
348
346
333
334
1
2
3
4
5
6
7
8
9
10
11
12
13
14
LEGEND
—350— CONTOUR LINE AND ELEVATION ABOVE SEA LEVEL IN FEET.
EXPLORATORY HOLE AND NUMBER.
EXPLORATORY HOLE FILLED AND COVERED OVER.
SCALE: 1" = 40'

GROUNDWATER CONTOURS
MARCH 30, 1984

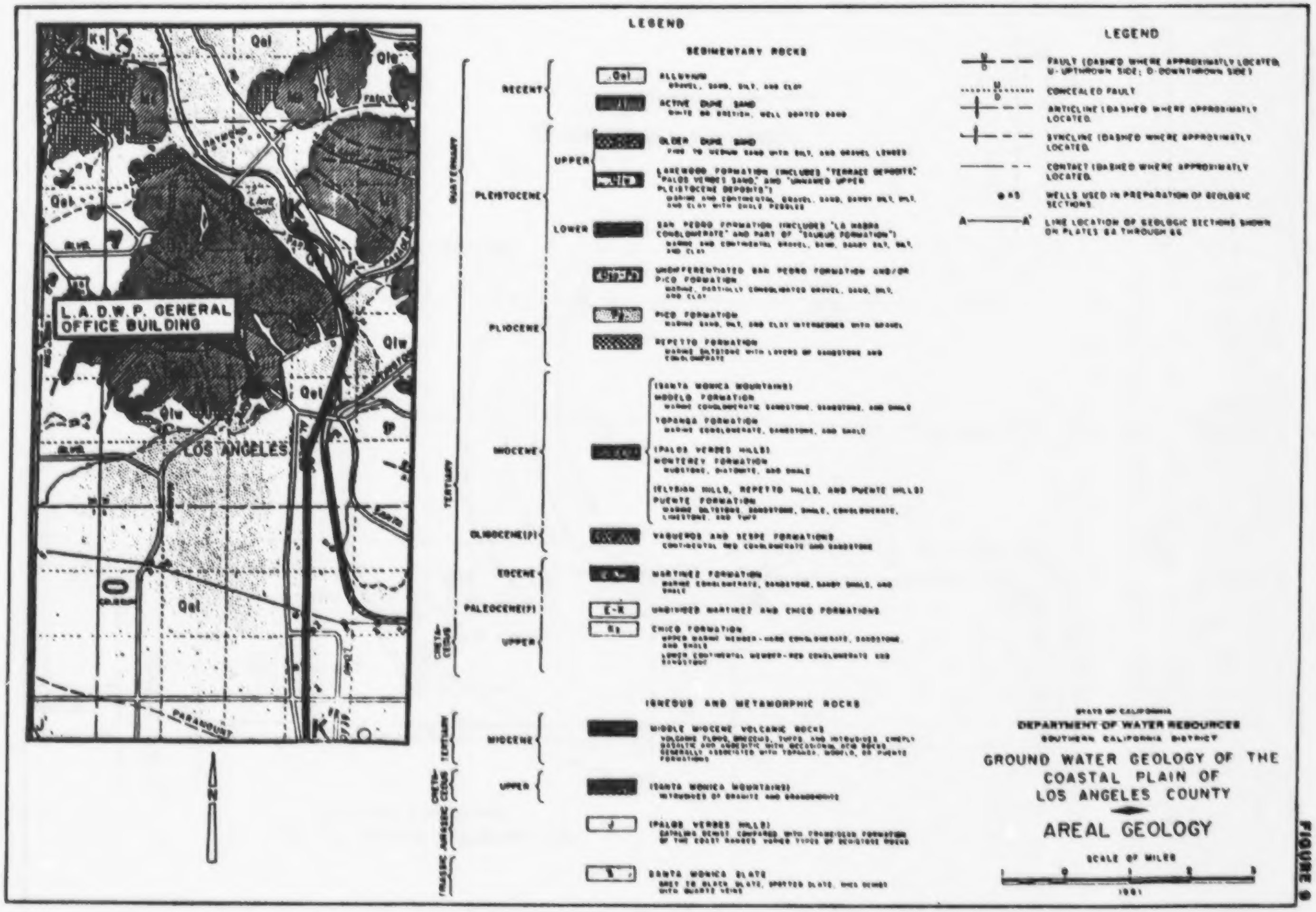
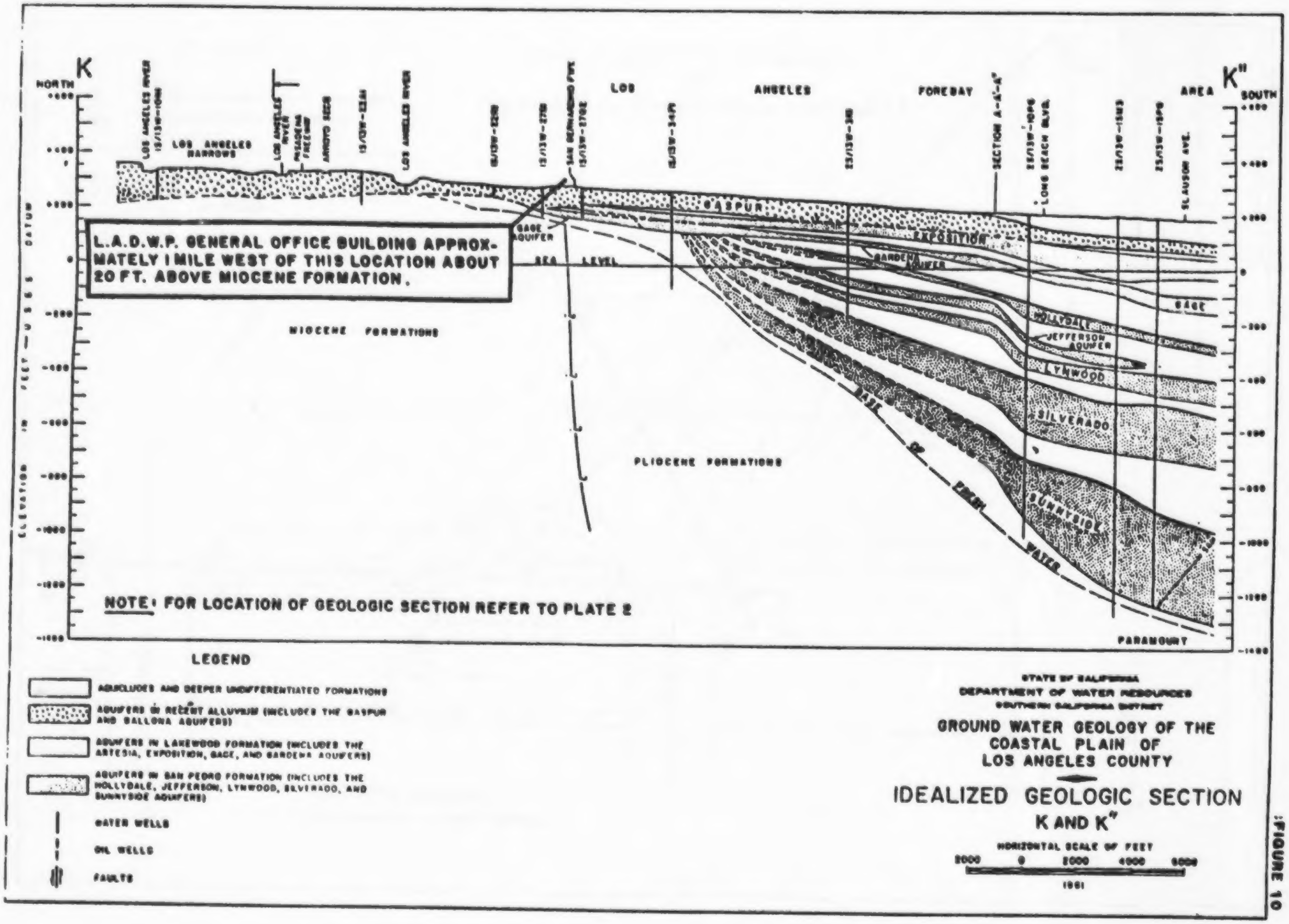

L.A.D.W.P. GENERAL OFFICE BUILDING
LOS ANGELES
Qal
Qlw
Kt
N
LEGEND
SEDIMENTARY ROCKS
QUATERNARY
RECENT
ALLUVIUM
GRAVEL, SAND, SILT, AND CLAY
ACTIVE DUNE SAND
WHITE OR BROWNISH, WELL SORTED SAND
PLEISTOCENE
UPPER
OLDER DUNE SAND
FINE TO MEDIUM SAND WITH SILT, AND GRAVEL LENSES
LAKEWOOD FORMATION (INCLUDES "TERRACE DEPOSITS," "PALOS VERDES SAND," AND "UNNAMED UPPER PLEISTOCENE DEPOSITS")
LOWER
SAN PEDRO FORMATION (INCLUDES "LA HABRA CONGLOMERATE" AND PART OF "SAUGUS FORMATION")
UNDIFFERENTIATED SAN PEDRO FORMATION AND/OR PICO FORMATION
PLIOCENE
PICO FORMATION
REPETTO FORMATION
TERTIARY
MIOCENE
(SANTA MONICA MOUNTAINS)
MODELO FORMATION
TOPANGA FORMATION
(PALOS VERDES HILLS)
MONTEREY FORMATION
(ELYSIAN HILLS, REPETTO HILLS, AND PUENTE HILLS)
PUENTE FORMATION
OLIGOCENE(?)
VAQUEROS AND SESPE FORMATIONS
EOCENE
MARTINEZ FORMATION
PALEOCENE(?)
UNDIVIDED MARTINEZ AND CHICO FORMATIONS
CRETACEOUS
UPPER
CHICO FORMATION
IGNEOUS AND METAMORPHIC ROCKS
TERTIARY
MIOCENE
MIDDLE MIOCENE VOLCANIC ROCKS
CRETACEOUS
UPPER
(SANTA MONICA MOUNTAINS)
JURASSIC
(PALOS VERDES HILLS)
TRIASSIC
SANTA MONICA SLATE
LEGEND
FAULT (DASHED WHERE APPROXIMATELY LOCATED, U-UPTHROWN SIDE; D-DOWNTHROWN SIDE)
CONCEALED FAULT
ANTICLINE (DASHED WHERE APPROXIMATELY LOCATED)
SYNCLINE (DASHED WHERE APPROXIMATELY LOCATED)
CONTACT (DASHED WHERE APPROXIMATELY LOCATED)
WELLS USED IN PREPARATION OF GEOLOGIC SECTIONS
LINE LOCATION OF GEOLOGIC SECTIONS SHOWN ON PLATES 6A THROUGH 6G
STATE OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
SOUTHERN CALIFORNIA DISTRICT
GROUND WATER GEOLOGY OF THE COASTAL PLAIN OF LOS ANGELES COUNTY
AREAL GEOLOGY
SCALE OF MILES
0
1
2
3
1961

FIGURE 9


NORTH
K
LOS ANGELES RIVER
LOS ANGELES NARROWS
LOS ANGELES RIVER
PASADENA FREEWAY
ARROYO SECO
LOS ANGELES RIVER
SAN BERNARDINO FWY
LOS
ANGELES
FOREBAY
AREA
SECTION A-A'-A"
LONG BEACH BLVD.
SLAUSON AVE.
K"
SOUTH
+600
+400
+200
0
-200
-400
-600
-800
-1000
-1200
-1400
ELEVATION IN FEET — U.S.G.S. DATUM
L.A.D.W.P. GENERAL OFFICE BUILDING APPROX-MATELY 1 MILE WEST OF THIS LOCATION ABOUT 20 FT. ABOVE MIOCENE FORMATION.
GAGE AQUIFER
SEA LEVEL
GASPUR
EXPOSITION
GARDENA AQUIFER
GAGE
HOLLYDALE
JEFFERSON AQUIFER
LYNWOOD
SILVERADO
SUNNYSIDE
BASE OF FRESH WATER
MIOCENE FORMATIONS
PLIOCENE FORMATIONS
PARAMOUNT
NOTE: FOR LOCATION OF GEOLOGIC SECTION REFER TO PLATE 2
LEGEND
AQUICLUDES AND DEEPER UNDIFFERENTIATED FORMATIONS
AQUIFERS IN RECENT ALLUVIUM (INCLUDES THE GASPUR AND BALLONA AQUIFERS)
AQUIFERS IN LAKEWOOD FORMATION (INCLUDES THE ARTESIA, EXPOSITION, GAGE, AND GARDENA AQUIFERS)
AQUIFERS IN SAN PEDRO FORMATION (INCLUDES THE HOLLYDALE, JEFFERSON, LYNWOOD, SILVERADO, AND SUNNYSIDE AQUIFERS)
WATER WELLS
OIL WELLS
FAULTS
HORIZONTAL SCALE OF FEET
2000
0
2000
4000
6000

STATE OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
SOUTHERN CALIFORNIA DISTRICT

GROUND WATER GEOLOGY OF THE
COASTAL PLAIN OF
LOS ANGELES COUNTY

IDEALIZED GEOLOGIC SECTION
K AND K"

1961

FIGURE 10

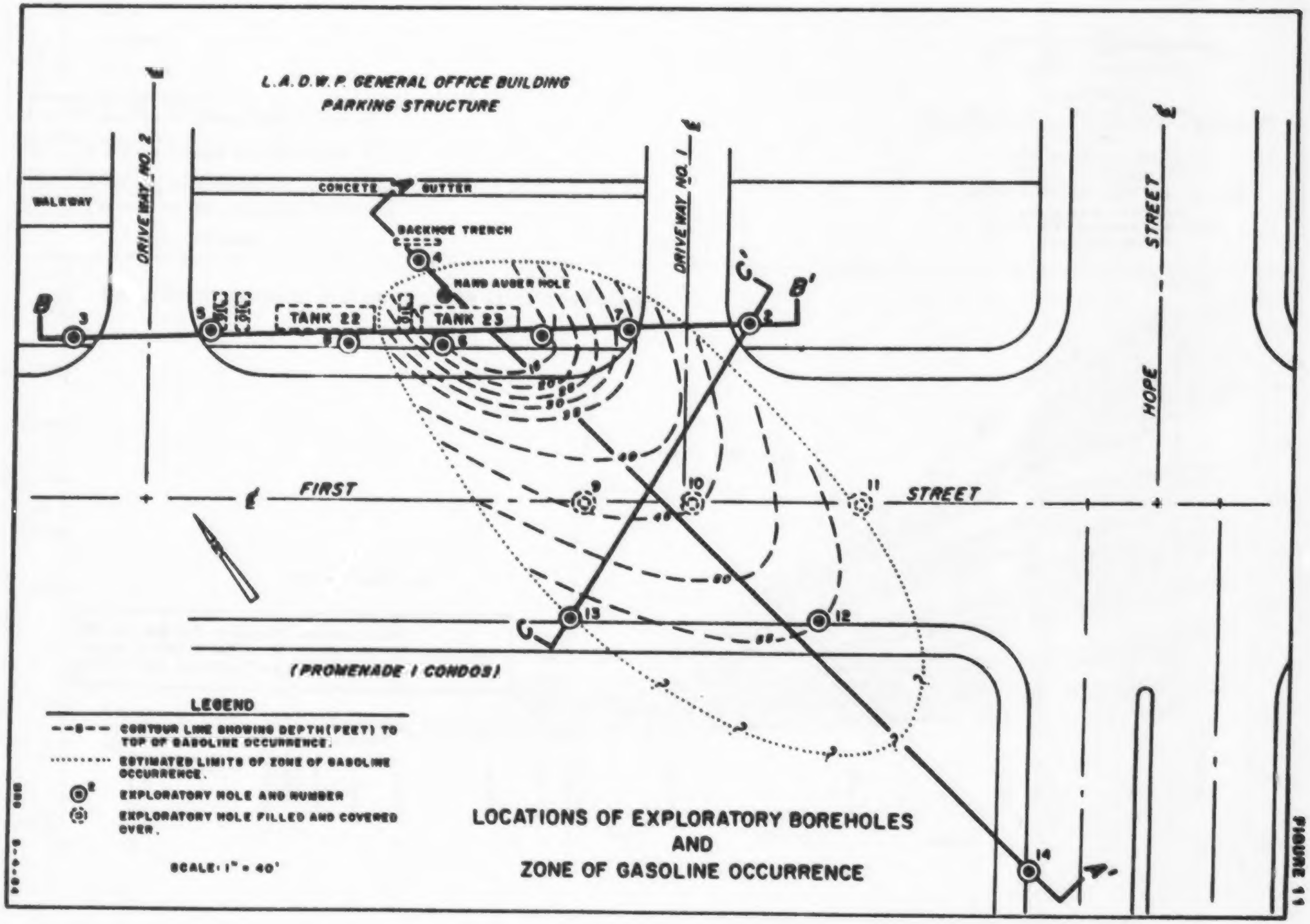
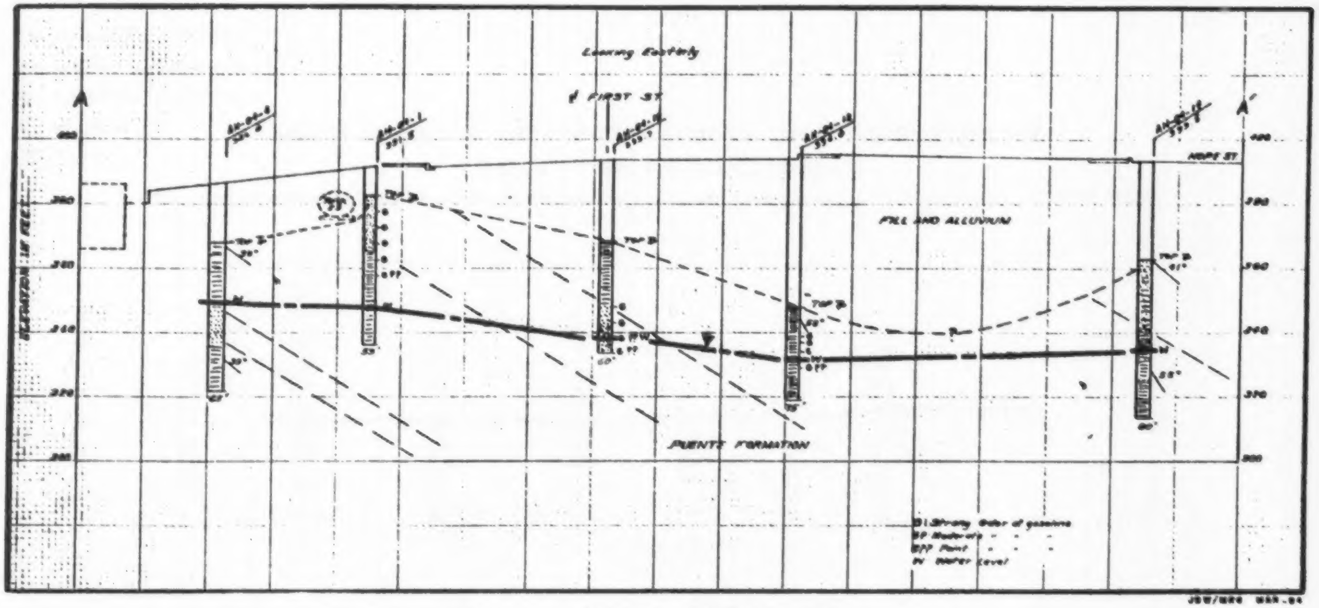

L. A. D. W. P. GENERAL OFFICE BUILDING
PARKING STRUCTURE
WALKWAY
DRIVEWAY NO. 2
DRIVEWAY NO. 1
CONCRETE GUTTER
BACKHOE TRENCH
HAND AUGER HOLE
TANK 22
TANK 23
B
B'
C
C'
A'
3
5
4
6
7
2
9
10
11
12
13
14
FIRST
STREET
HOPE
STREET
(PROMENADE I CONDOS)
10
20
30
40
50
?
LEGEND
CONTOUR LINE SHOWING DEPTH (FEET) TO TOP OF GASOLINE OCCURRENCE.
ESTIMATED LIMITS OF ZONE OF GASOLINE OCCURRENCE.
EXPLORATORY HOLE AND NUMBER
EXPLORATORY HOLE FILLED AND COVERED OVER.
SCALE: 1" = 40'
SRD 8-4-84

FIGURE 11

LOCATIONS OF EXPLORATORY BOREHOLES
AND
ZONE OF GASOLINE OCCURRENCE


Looking Easterly
℄ FIRST ST
A
A'
HOPE ST
FILL AND ALLUVIUM
PUENTE FORMATION
ELEVATION IN FEET
400
380
360
340
320
300
S : Strong Odor of gasoline
SP Moderate " "
SPP Faint " "
W Water Level
JSW/GRG MAR. 84

L.A.D.W.P. - GENERAL OFFICE BUILDING

CROSS SECTION A – A' THROUGH ZONE OF GASOLINE OCCURRENCE

▤ SHALE

▦ SANDSTONE

S : STRONG ODOR OF GASOLINE

SP : MODERATE ODOR OF GASOLINE

SPP: FAINT ODOR OF GASOLINE

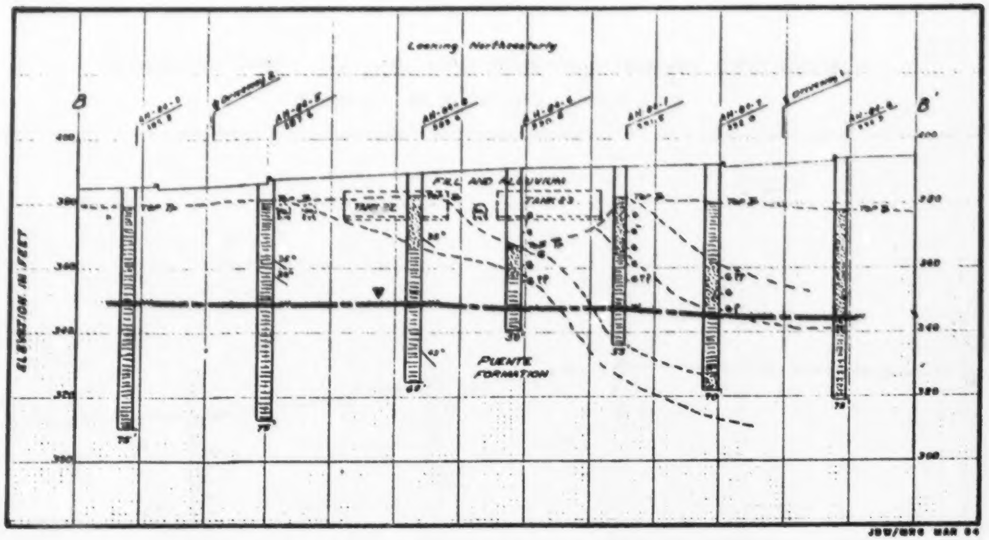
W : WATER LEVEL ———

T_p : TERTIARY PUENTE FORMATION

$\frac{\text{AH-84-4}}{\text{388.8}}$: $\frac{\text{AUGER HOLE - YEAR DRILLED - NUMBER}}{\text{GROUND SURFACE ELEVATION - FEET}}$

NOTE : REFER TO PLATE 8 FOR LOCATION OF CROSS SECTION.

FIGURE 12



Looking Northeasterly
B
B'
Driveway 2
Driveway 1
400
380
360
340
320
300
ELEVATION IN FEET
FILL AND ALLUVIUM
TANK 22
TANK 23
Top Tp
PUENTE FORMATION
75'
75'
65'
30'
35'
70'
75'
JBW/MRG MAR 84

L.A.D.W.P. - GENERAL OFFICE BUILDING

CROSS SECTION B – B' THROUGH ZONE OF GASOLINE OCCURRENCE

▥ SHALE

▦ SANDSTONE

G : STRONG ODOR OF GASOLINE

GF : MODERATE ODOR OF GASOLINE

GFF : FAINT ODOR OF GASOLINE

W : WATER LEVEL ———

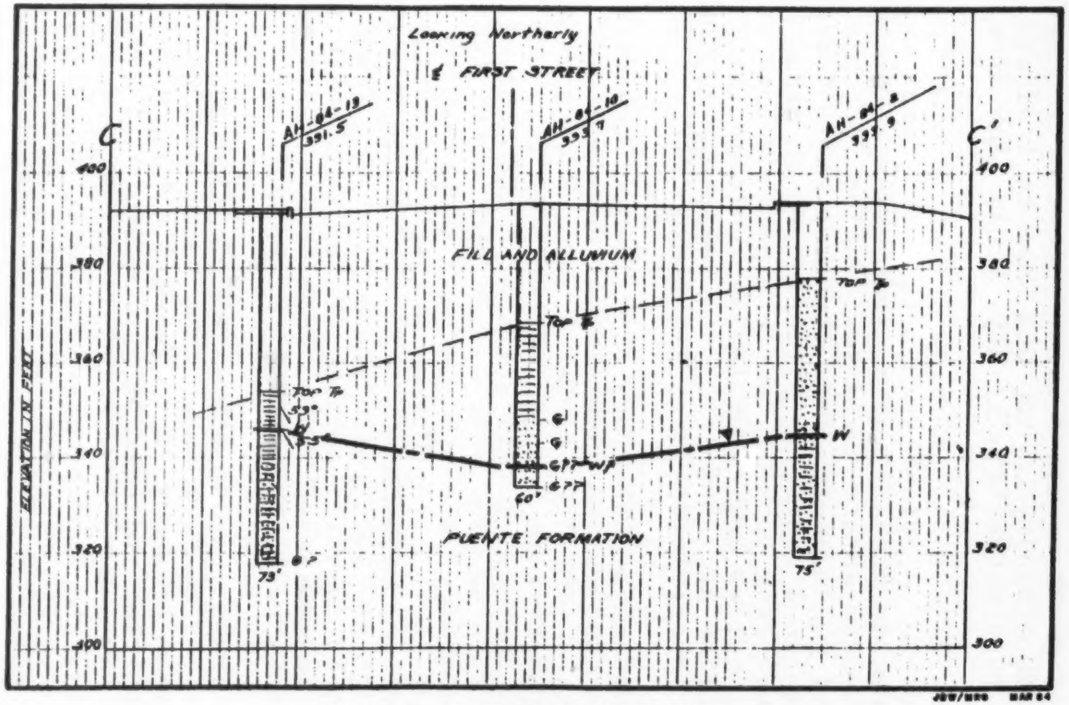
Tp : TERTIARY PUENTE FORMATION

AH - 84 - 4 / 386.6 : AUGER HOLE - YEAR DRILLED - NUMBER / GROUND SURFACE ELEVATION - FEET

NOTE : REFER TO PLATE 3 FOR LOCATION OF CROSS SECTION.

FIGURE 13

FIGURE 14


Looking Northerly
℄ FIRST STREET
C
C'
AH-84-13
391.5
AH-84-10
393.7
AH-84-8
393.9
400
380
360
340
320
300
ELEVATION IN FEET
FILL AND ALLUVIUM
Top Tp
59'
W
53'
G
G
GPP WP
GPP
60'
73'
GP
75'
PUENTE FORMATION
JBW/HRG MAR 84

L.A.D.W.P - GENERAL OFFICE BUILDING

CROSS SECTION C – C' THROUGH ZONE OF GASOLINE OCCURRENCE

▤ SHALE

▦ SANDSTONE

G : STRONG ODOR OF GASOLINE

GP : MODERATE ODOR OF GASOLINE

GPP: FAINT ODOR OF GASOLINE

W : WATER LEVEL ——

T_p : TERTIARY PUENTE FORMATION

AH-84-4 / 388.8 : AUGER HOLE - YEAR DRILLED - NUMBER / GROUND SURFACE ELEVATION - FEET

NOTE: REFER TO PLATE 3 FOR LOCATION OF CROSS SECTION.

The study site is at the southern edge of the Elysian Hills. A thin layer of alluvium and fill material overlies the bedrock formation. Recharge to the area comes from rainfall and irrigation. Rainfall averages about 15 inches annually. Much of the area around the GOB is paved over or covered in a way that limits the ground surface available for infiltration. Some water percolates vertically to the less permeable zones of the Formation, where dipping planes and low permeability cause the groundwater to flow in a more horizontal direction (Figures 11 to 14). The limited amount of recharge and the geology in the vicinity of the GOB seem to indicate that only a small amount of groundwater, if any, would flow from the study area.

The groundwater gradient within the study area was not previously defined due to the lack of monitor wells. Fourteen test holes were installed during the investigation for this purpose. Water level measurements from the test wells indicate the groundwater flows in a southerly direction to an area approximately 150 feet from Tank 23, where a perched zone is encountered. Drilling logs and water levels indicate that the groundwater and gasoline are contained by a bowl-shaped channel of low permeability materials (Figures 11 through 14).

The groundwater at the study site is not considered to be of high enough quality and quantity for developing as a source of water supply. The low volume of perched groundwater that exists in the Puente Formation is probably poor in quality due to the long-time contact of this water with the high-clay-content formation. This is confirmed by the poor water quality immediately down-gradient of the study area. According to Bookman and Edmonston (1970), the Gaspur Aquifer has suffered substantial water quality degradation. Total dissolved solids concentrations as high as 1,000 parts per million are found in shallow aquifer. No groundwater pumping occurs in the vicinity of the study site; the nearest production wells are nearly five miles away, to the southeast, in the City of Vernon.

The field investigation consisted of drilling, logging, and soil sampling 14 hollow-stem auger test holes (wells). The wells were drilled from February 22 to March 8, 1984, under the supervision of a certified engineering geologist, with hydrologic engineers providing advisory guidance. Soil samples were taken, examined in the field, and submitted to the water quality lab for analysis.

The geologist on site during drilling examined the soil samples by smelling for gasoline. Immediately after collecting the samples, they were put in air tight containers for further analysis by the water quality lab. Several samples were tested by gas chromatograph mass spectrometer (GCMS) analysis. The GCMS analyzes the vapors from the soil sample and identifies several individual compounds present in the gasoline. The GCMS was also used to examine a sample of gasoline from the garage for comparison with a gasoline sample obtained from Well 12. The sample from Well 12 contained several of the same compounds found in the garage gas sample.

The geology and the intervals of gasoline occurrence (as determined from the soil samples) are shown on Figures 12 through 14. Figure 11 shows the probable limits of the gasoline and contours of depth from ground surface to the top of the gasoline zone. The area of gasoline occurrence is estimated to be about 225 feet long and 100 feet wide, based on contours drawn from information obtained by drilling. The top of the gasoline zone is from 15 feet deep, near the tank, to 55 feet deep at the south curb of First Street.

A backhoe trench was excavated adjacent to Well 4 to a depth of about eight feet. A small portion of the gasoline appears to have migrated to the north of the tank, where some of it may have entered the sand backfill at the side of the basement garage. The odor of stale (old or residual) gasoline was observed in the upper four feet of the trench; however, test Well #4 (drilled near the trench area) did not encounter any gasoline from 10 feet to 65 feet.

Clean-up of the gasoline that had migrated to the various monitoring wells (primarily Well 12) was started on May 8, 1984. The clean-up is being accomplished by hand bailing of each well containing gasoline. The hand bailer consists of a two-foot long, 2½-inch diameter galvanized steel pipe, capped on the bottom, and wrapped with insulation tape (to prevent sparks). The bailer is lowered and raised by a rope and pully. The gasoline and water collected in each well is measured, then discharged into a 55 gallon drum and filled with soil. The drums are then transported to the BKK, Class I, landfill in Covina.

Based on monitoring of the water and gasoline levels before and after each bailing session in Well No. 12, the gasoline level in the casing recovers to approximately 90 percent of the prebailing level in a 24-hour period. Observation of levels in Well No. 1 show that the gasoline continues to fill in after bailing, similar to Well No. 12, but at a much slower rate. Indications are that the tightness of the Puente Formation limits the ability to extract the free gasoline by normal pumping techniques. Thus, the bailing or small volume pumping (planned for the near future) will have to be continued for an indefinite period of time.

The migration of gasoline appears to have been contained or at least limited due to the geologic formation. The gasoline appears to be collecting in the vicinity of Well No. 12 (see Figures 11 and 12). Thus, the clean-up procedure being used is as follows:

Clean-Up Procedures

1. The Casing of wells where gasoline is present are evacuated on a regular basis (the interval may vary

with time depending on the volume of gasoline retrieved).

2. The gasoline-free perimeter wells are being monitored on a regular basis to verify that the gasoline has not migrated to those wells.

3. Tank 23 will be removed and replaced with one that meets current regulatory requirements.

It is believed that the bailing of the wells will eventually remove most of the free gasoline from the formation. Clean-up by bailing at regular intervals is necessary, as it is very unlikely that an operation to develop a cone of depression by pumping can be accomplished.

In addition, studies made after the 1968 Western Oil and Gas Association gasoline pipe leakage problem in the Forest Lawn area of Glendale (in the San Fernando Valley) have shown that naturally occurring, but normally dormant bacteria (principally genus Pseudomonas) which biologically break-down hydrocarbons, will eventually clean the saturated and unsaturated portions of the Formation of residual gasoline.

Gasoline Tank Leak - LADWP, South Los Angeles Yard

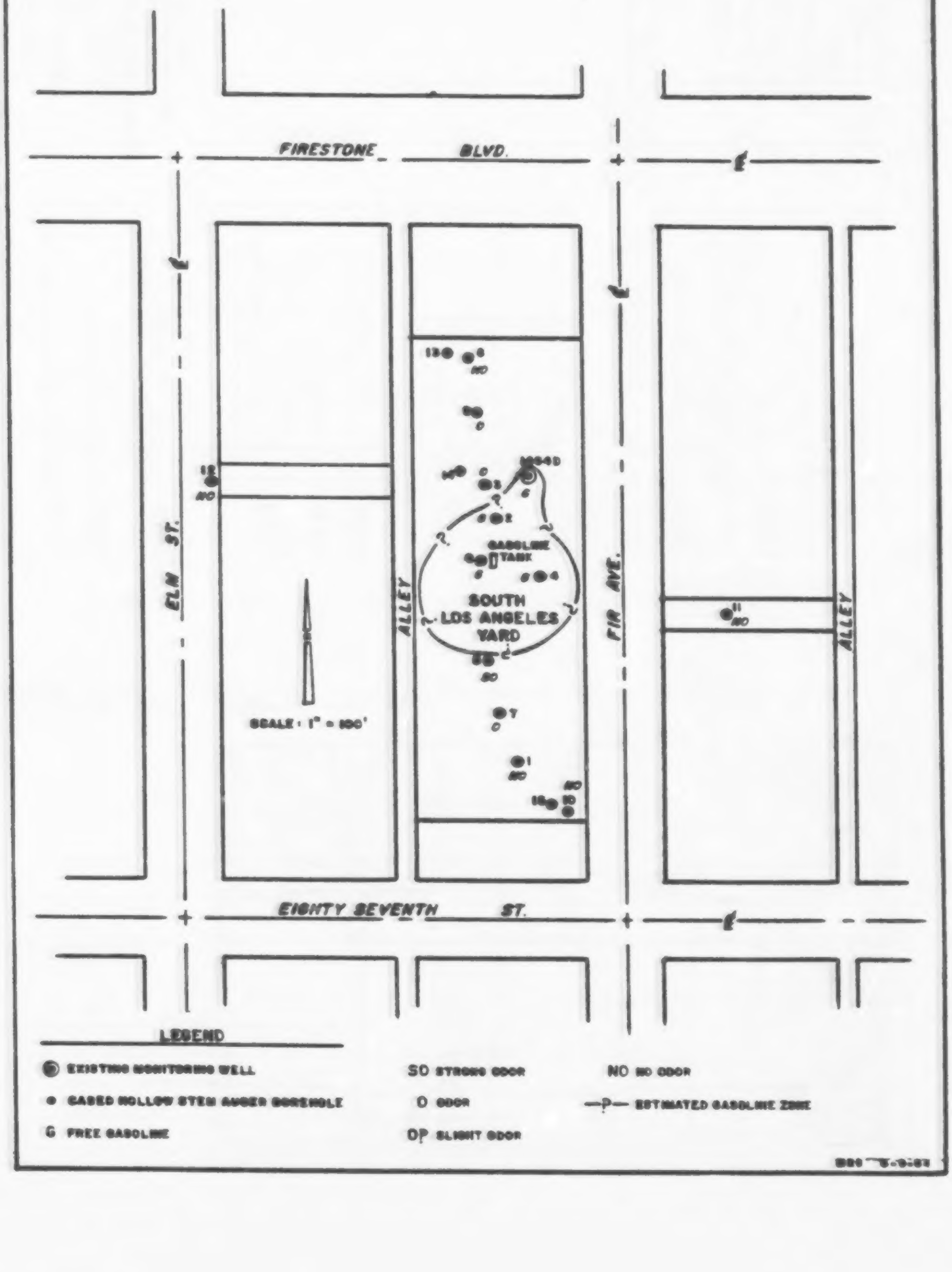
A gasoline tank leak was discovered by the LADWP on March 20, 1984 upon removal of an existing 1,000 gallon steel underground gasoline storage tank which had been scheduled for replacement. Inspection of the tank revealed several small holes in the tank wall apparently as a result of corrosion. Soil samples taken from beneath the tank were found to be contaminated with gasoline, but it is not known what quantity of gasoline was lost. Boreholes (wells) were drilled to define the horizontal and vertical limits of the gasoline migration, and to develop a clean-up plan (see Figure 15).

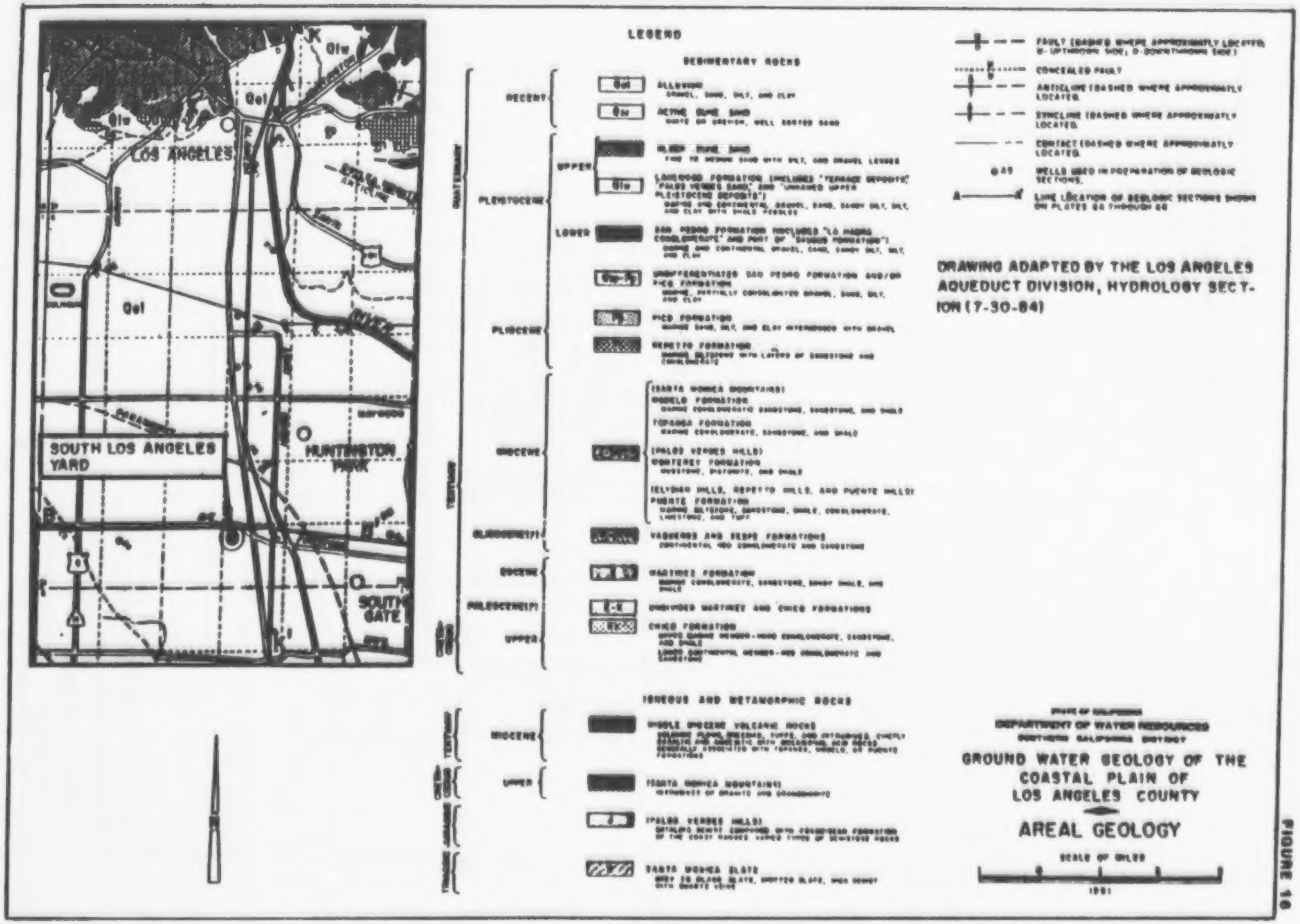
The geology in the area of the South LA Yard is recent alluvium consisting of gravel, sand, silt, and clay (see Figure 16). This was verified when 12 shallow and 3 deep hollow-stem auger holes (wells) were drilled and logged at the South LA Yard. The permeability of the fine grained alluvium overlying the silty and clayey formation is relatively low, as indicated by perched groundwater at approximately 35 feet (see Figures 19 through 22). The zones of low permeability and the occurrence of perched groundwater restrict vertical migration of the gasoline. The main water-bearing formation was reached at a depth of approximately 85 feet (Figure 21). Cross sections B-B' (Figure 17) depicts the general geology of the area.

A geologic section (Figure 17) depicting the groundwater hydrology of the South LA Yard area was previously prepared by the California Department of Water Resources (1961). The study site is located within the Central Basin of the Coastal Plain, and very

FIGURE 15

LOCATIONS OF EXPLORATORY BOREHOLES FOR GASOLINE TANK LEAK AT SOUTH LOS ANGELES YARD


FIRESTONE BLVD.
EIGHTY SEVENTH ST.
ELM ST.
FIR AVE.
ALLEY
ALLEY
SOUTH LOS ANGELES YARD
GASOLINE TANK
SCALE: 1" = 100'
NO
D
SO
G
LEGEND
EXISTING MONITORING WELL
CASED HOLLOW STEM AUGER BOREHOLE
G FREE GASOLINE
SO STRONG ODOR
O ODOR
OP SLIGHT ODOR
NO NO ODOR
—P— ESTIMATED GASOLINE ZONE



Qlu
Qal
LOS ANGELES
RIVER
HUNTINGTON PARK
SOUTH LOS ANGELES YARD
SOUTH GATE
LEGEND
SEDIMENTARY ROCKS
RECENT
PLEISTOCENE
UPPER
LOWER
PLIOCENE
MIOCENE
OLIGOCENE(?)
EOCENE
PALEOCENE(?)
UPPER
QUATERNARY
TERTIARY
Qal ALLUVIUM
Qsu ACTIVE DUNE SAND
PICO FORMATION
REPETTO FORMATION
MODELO FORMATION
TOPANGA FORMATION
MONTEREY FORMATION
PUENTE FORMATION
VAQUEROS AND SESPE FORMATIONS
MARTINEZ FORMATION
UNDIVIDED MARTINEZ AND CHICO FORMATIONS
CHICO FORMATION
IGNEOUS AND METAMORPHIC ROCKS
MIOCENE
UPPER
MIDDLE MIOCENE VOLCANIC ROCKS
SANTA MONICA SLATE
CONCEALED FAULT
ANTICLINE (DASHED WHERE APPROXIMATELY LOCATED)
SYNCLINE (DASHED WHERE APPROXIMATELY LOCATED)
CONTACT (DASHED WHERE APPROXIMATELY LOCATED)
WELLS USED IN PREPARATION OF GEOLOGIC SECTIONS.
SCALE OF MILES
1961

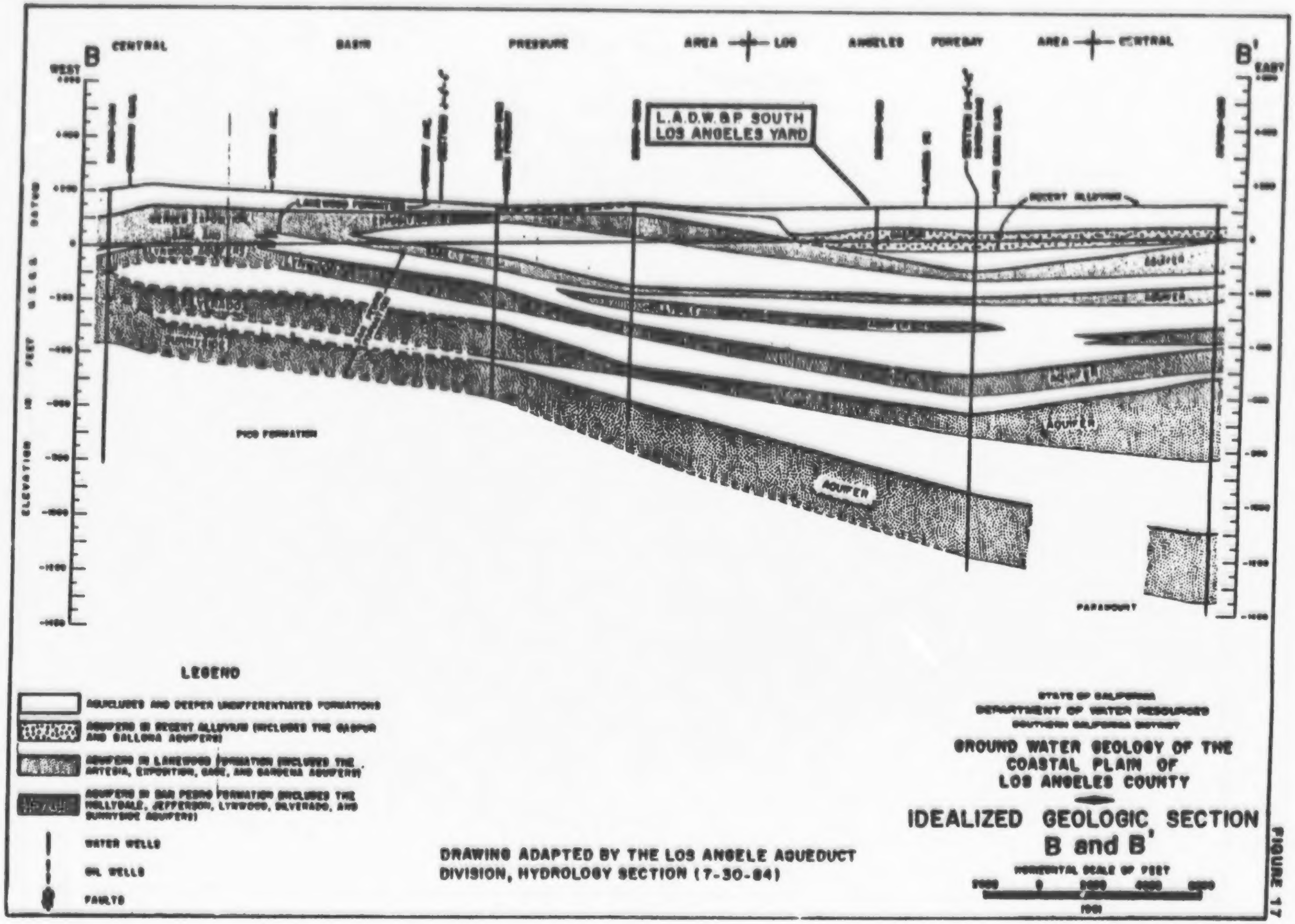
DRAWING ADAPTED BY THE LOS ANGELES AQUEDUCT DIVISION, HYDROLOGY SECTION (7-30-84)

STATE OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
SOUTHERN CALIFORNIA DISTRICT

GROUND WATER GEOLOGY OF THE COASTAL PLAIN OF LOS ANGELES COUNTY

AREAL GEOLOGY

FIGURE 16


B
B'
WEST
EAST
CENTRAL
BASIN
PRESSURE
AREA
LOS
ANGELES
FOREBAY
AREA
CENTRAL
L.A.D.W.&P SOUTH
LOS ANGELES YARD
RECENT ALLUVIUM
AQUIFER
AQUIFER
PICO FORMATION
PARAMOUNT
ELEVATION IN FEET U.S.G.S. DATUM
0
LEGEND
AQUICLUDES AND DEEPER UNDIFFERENTIATED FORMATIONS
AQUIFERS IN RECENT ALLUVIUM (INCLUDES THE GASPUR AND BALLONA AQUIFERS)
AQUIFERS IN LAKEWOOD FORMATION (INCLUDES THE ARTESIA, EXPOSITION, GAGE, AND GARDENA AQUIFERS)
AQUIFERS IN SAN PEDRO FORMATION (INCLUDES THE HOLLYDALE, JEFFERSON, LYNWOOD, SILVERADO, AND SUNNYSIDE AQUIFERS)
WATER WELLS
OIL WELLS
FAULTS
HORIZONTAL SCALE OF FEET
2000
0
2000
4000
6000
1961

DRAWING ADAPTED BY THE LOS ANGELE AQUEDUCT DIVISION, HYDROLOGY SECTION (7-30-84)

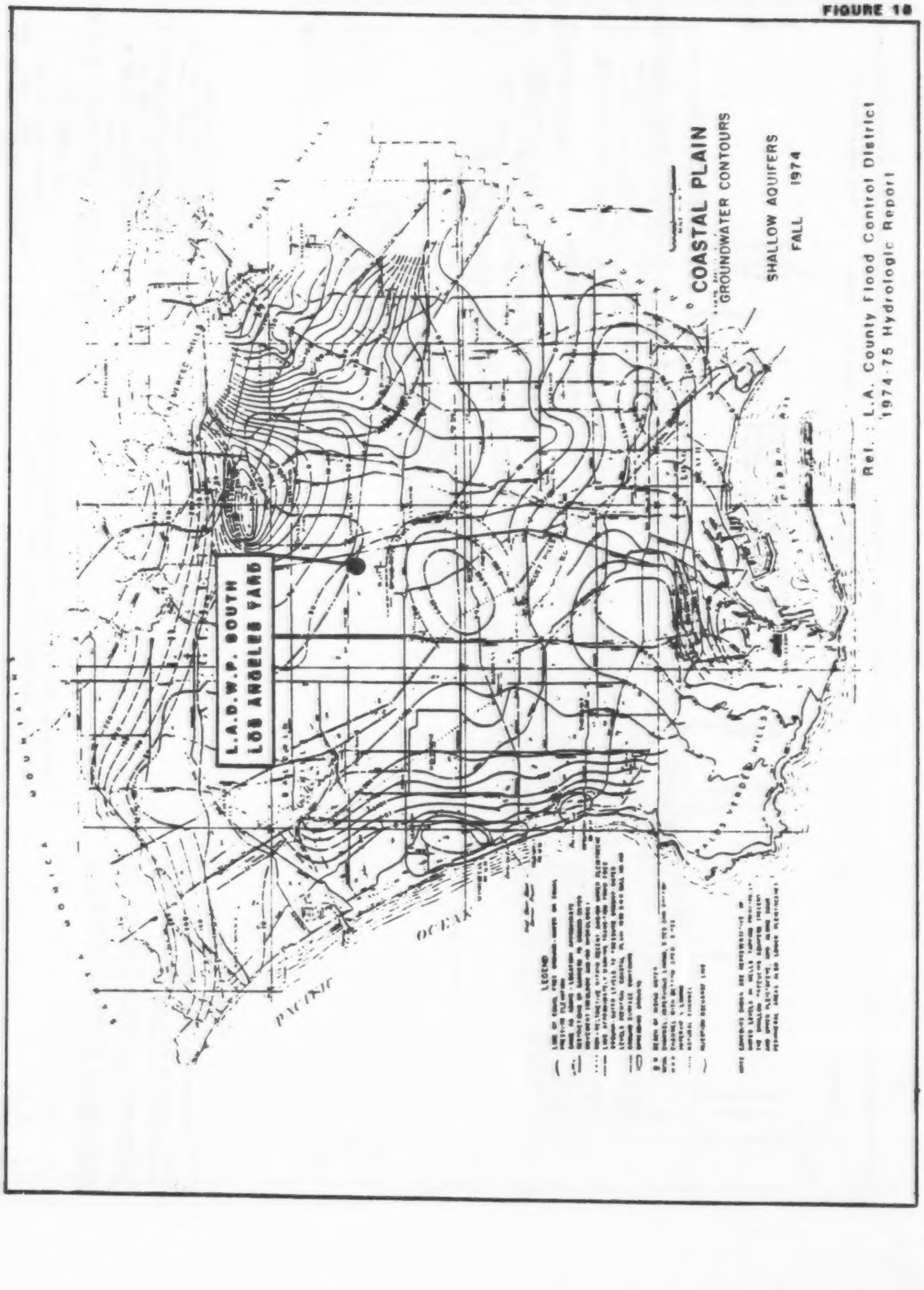
STATE OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
SOUTHERN CALIFORNIA DISTRICT

GROUND WATER GEOLOGY OF THE
COASTAL PLAIN OF
LOS ANGELES COUNTY

IDEALIZED GEOLOGIC SECTION
B and B'

FIGURE 17

FIGURE 18


COASTAL PLAIN
GROUNDWATER CONTOURS
SHALLOW AQUIFERS
FALL 1974
L.A.D.W.P. SOUTH
LOS ANGELES YARD
PACIFIC
OCEAN
LEGEND

Ref. : L.A. County Flood Control District
1974-75 Hydrologic Report

FIGURE 19

LOCATIONS OF CROSS SECTIONS A-A', B-B' AND C-C'

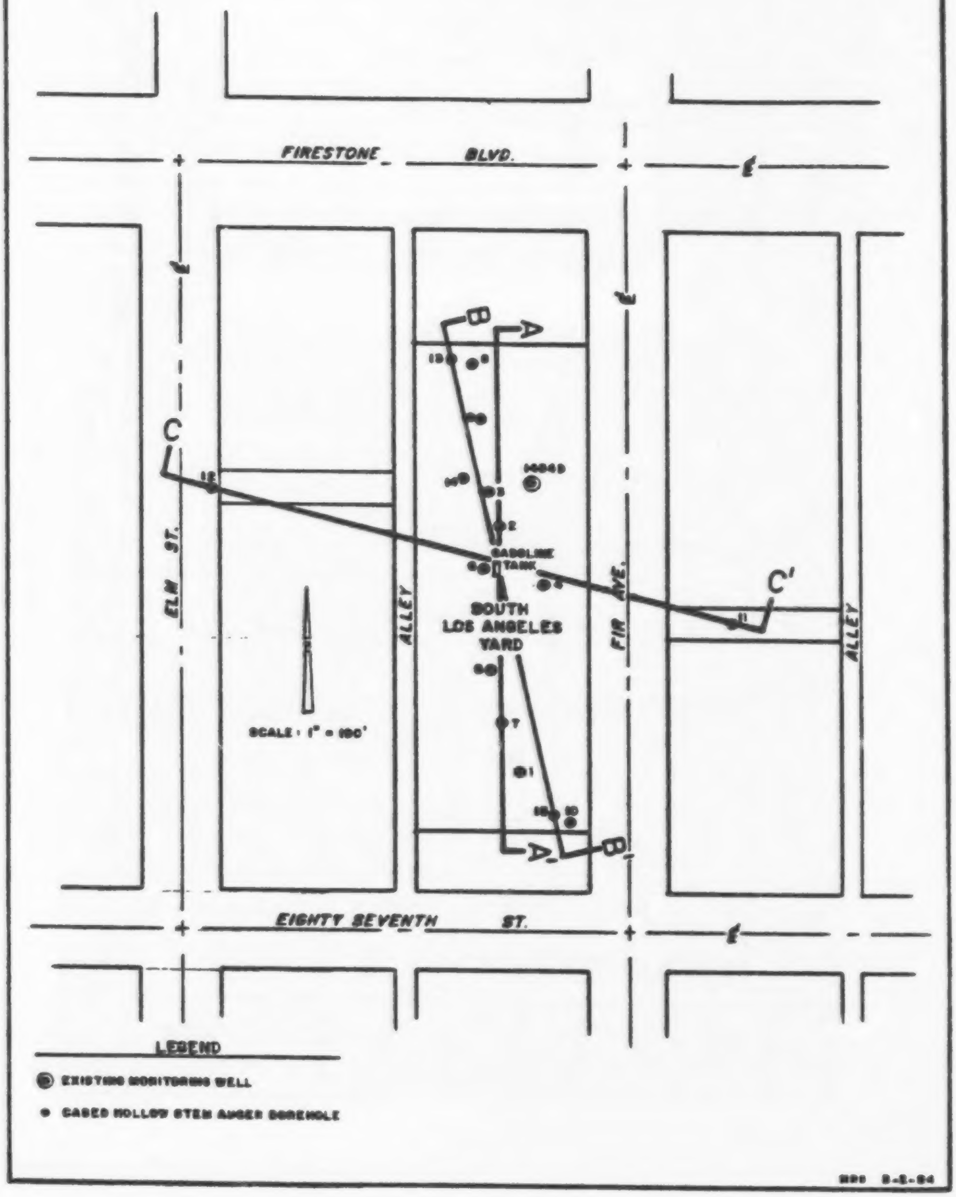
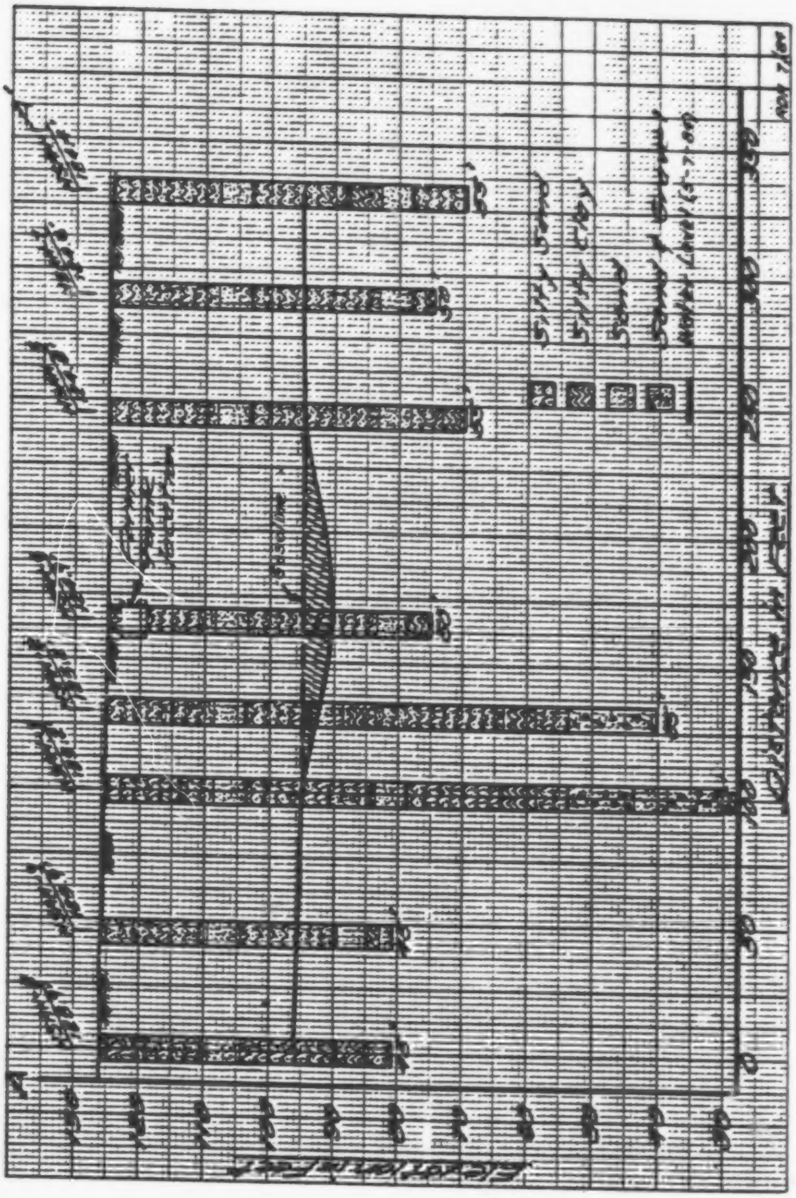
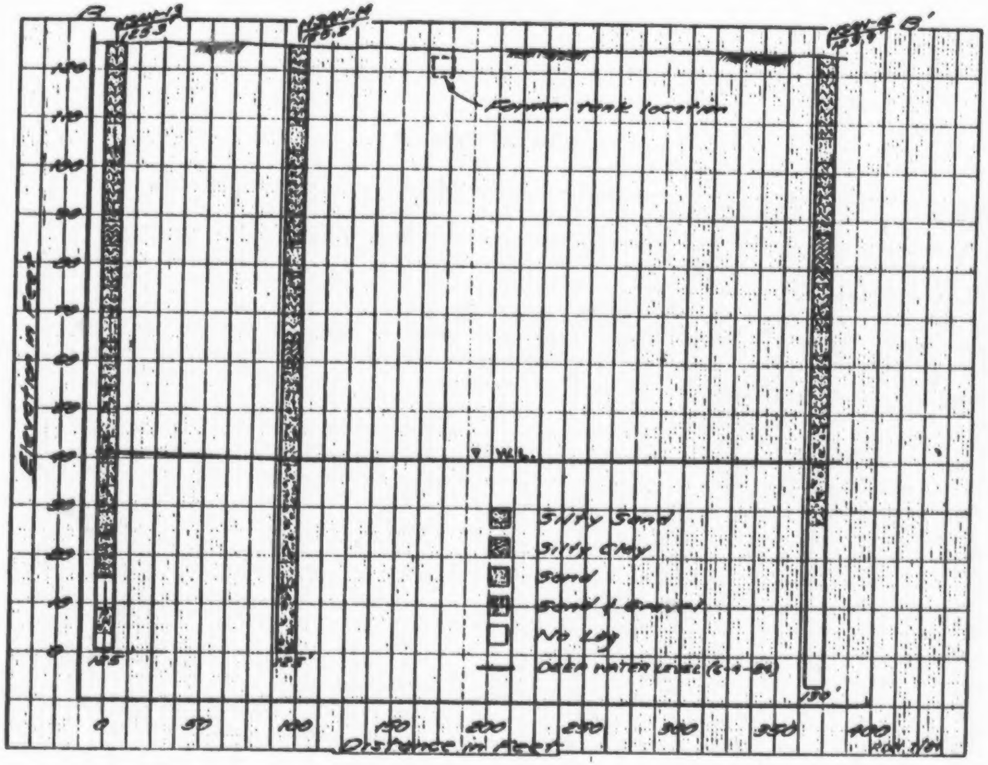

FIRESTONE BLVD.
EIGHTY SEVENTH ST.
ELM ST.
FIR AVE.
ALLEY
ALLEY
A
A'
B
B'
C
C'
13
3
2
GASOLINE TANK
SOUTH LOS ANGELES YARD
7
1
10
12
11
SCALE: 1" = 100'
LEGEND
EXISTING MONITORING WELL
CASED HOLLOW STEM AUGER BOREHOLE

FIGURE 20

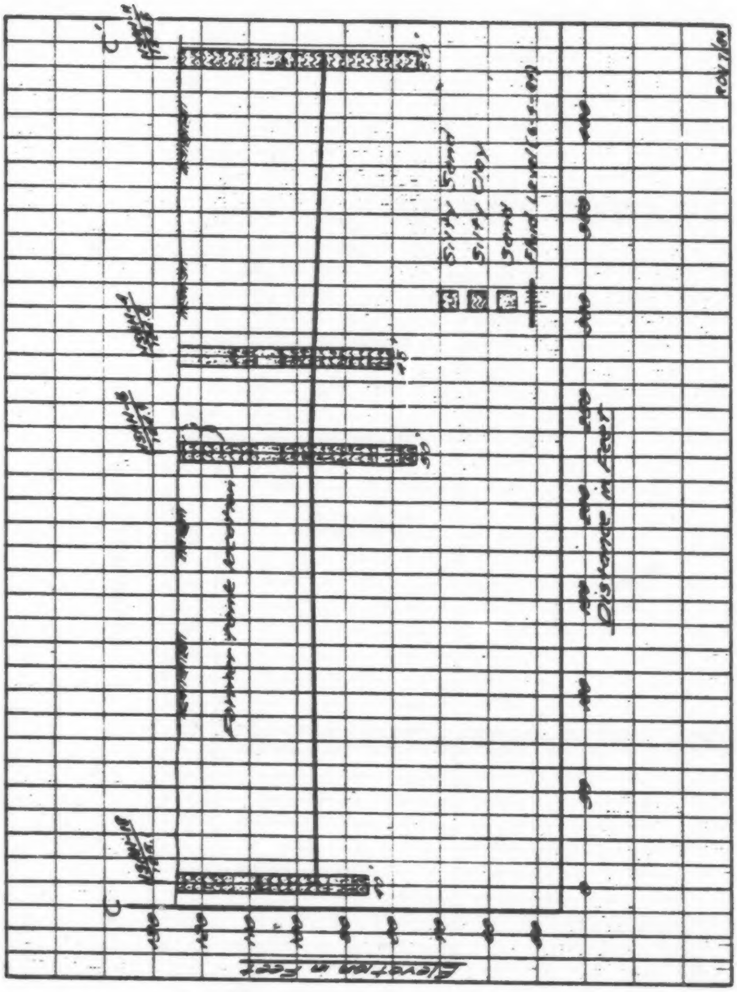

A
A'
Silty Sand
Silty Clay
Sand
Sand & Gravel
Water Level (2-7-80)
0
50
100
150
200
250
300
350
Distance in Feet
Elevation in Feet

CROSS SECTION A - A'


B
B'
Former tank location
Elevation in Feet
120
110
100
90
80
70
60
50
40
30
20
10
0
W.L.
Silty Sand
Silty Clay
Sand
Sand & Gravel
No Log
Observation Well 125', 125', 130' total depth
Distance in Feet
0
50
100
150
200
250
300
350
400

CROSS SECTION B - B'

FIGURE 22



C
C'
Former pond location
Silty Sand
Silty Clay
Sand
Fluid Level (6-9-81)
Distance in Feet
Elevation in Feet

CROSS SECTION C - C'

close to the pressure, non-pressure boundary within the Basin (see Figure 18).

The perched groundwater occurs as percolating water through the higher permeability alluvium and fill material overlying clayey strata (aquiclude), while the main (deeper) water-bearing formation is supplied by some underflow from the San Gabriel Valley and from water that percolates into the non-pressure area of the Basin.

Recharge to the area comes from rainfall and irrigation. Rainfall averages about 15 inches annually. Much of the area around the South LA Yard is paved over or covered in a way that limits the ground surface available for infiltration. Some water percolates vertically to the less permeable zones of the formation, where it then flows in a more horizontal direction (Figure 18). The limited amount of recharge and the geology in the vicinity of the South LA Yard indicates that no groundwater in the shallow zone would flow from the study area.

Fifteen test holes, twelve shallow and three deep, were installed during the investigation to define the groundwater gradient within the study area. Water level measurements from the twelve shallow test wells indicate the shallow groundwater gradient is quite flat with water possibly flowing in a southerly or southeasterly direction. Water level measurement from the three deep test wells and the one existing deep well indicate that the main groundwater flow is southwesterly (see Figure 21).

The field investigation consisted of drilling, logging, and soil sampling 15 hollow-stem auger test holes. The holes were drilled from April 16 to May 4, 1984, under the supervision of a certified engineering geologist, with a registered hydrologic engineer providing advisory guidance.

The geologist examined the soil samples by smelling for gasoline. The samples were then placed in air tight containers and delivered to the water quality lab where an odor panel analyzed each sample for the presence of gasoline.

The geology and the intervals of gasoline occurrence (as determined from the soil samples) are shown on Figure 20. Figure 15 shows the probable limits of the gasoline and the odor criteria for the shallow wells. The area of gasoline occurrence is estimated to be about 150 feet long and 125 feet wide, based on comparing water and fluid surface levels and by information obtained from drilling. Samples collected in the observation wells closest to the source indicate up to several feet of gasoline floating on the upper perched groundwater. Wells 8, 9, 10, 11, 13, and 15 are essentially free of gasoline. The water quality laboratory personnel detected minor amounts of gasoline is samples taken from the upper 20 feet of Well 12, although no odor was noticed by the geologist on site. A slight gasoline odor was detected by water quality laboratory personnel in samples from two exploratory holes below 50 feet (at 85-90 feet in

Well 13, and 100-110 feet in Well 14). Subsequent monitoring of these observation wells has indicated no detectable gasoline.

A former Conservative Water District production well (Well 1464D) which was drilled to a depth of 220 feet in 1906, is now being used as a groundwater observation well. The depth to water in this well is approximately 85 feet, with several feet of free gasoline observed floating on top. Indications are that gasoline from the leaking tank migrated laterally on the perched groundwater to the gravel pack of this deeper well, and entered the casing through holes in the casing caused by deterioration of the casing with time.

Clean-up of the gasoline that had migrated to the various monitoring wells (primarily Wells 2, 4, 6, and 1464D) started on May 23, 1984. Presently, the clean-up is being accomplished by hand bailing of each well containing gasoline. The hand bailer consists of a two-foot long 2½-inch diameter galvanized steel pipe, capped on the bottom, and wrapped with insulation tape (to prevent sparks). The bailer is lowered and raised by a rope and pulley. After measuring, the gasoline and water collected in each well is discharged into a 55 gallon drum. The drum is filled with soil and transported to the BKK, Class I, landfill in Covina. This is the same operation as used in the GOB operation.

Wells 2, 4, 6, and 1464D were the only wells to contain a significant amount of free gasoline. During the period from May 23 to August 30, a total of 390 gallons of gasoline was bailed from these wells. Initially, a thin film of gasoline was observed floating on water bailed from Wells 3 and 5. This film was removed and did not reappear during later bailing sessions.

Based on monitoring of the water and gasoline levels before and after bailing sessions on June 4 and June 6 in Wells 2, 4, and 6, the gasoline level in the casings recover to approximately 75 percent of the prebailing level in a 45-hour period. The tightness of the formation limits the ability to extract the free gasoline by normal pumping techniques. Thus, bailing will have to be continued for an indefinite period of time.

Migration of gasoline appears to have been contained due to the geologic formation and flat gradient. The gasoline pool is remaining in the vicinity of Wells 2, 4, and 6 (see Figures 15 and 20). Thus, the clean-up procedure being used is as follows:

Clean Up Procedures:

1. The casing of wells where gasoline is present are evacuated on a regular basis (the interval may vary with time depending on the volume of gasoline retrieved).

2. The gasoline-free perimeter wells are being monitored on a regular basis to verify that the gasoline has not migrated to those wells.

It is believed that the bailing of the wells will eventually remove most of the free gasoline from the formation. Clean-up by bailing at regular intervals is necessary and will continue because an operation to develop a cone of depression by pumping can probably not be accomplished.

In addition, studies made after the 1968 Western Oil and Gas Association gasoline pipe leakage problem in the Forest Lawn area of Glendale (in the San Fernando Valley) have shown that naturally occurring, but normally dormant bacteria (principally Pseudomonas) which biologically breakdown hydrocarbons, will eventually clean the saturated and unsaturated portion of the formation of residual gasoline.

PROPOSED GASOLINE LEAK CLEANUP PROCEDURES

Based on over 15 years of history regarding gasoline leaks and movement in groundwater reservoirs, a positive outlook in surfacing regarding restoring the contaminated aquifers to their original use. The restoration process involves three distinct phases:

1 - Detection and containment

2 - Removal of free gasoline

3 - Biodegradation of residual gasoline

As described in the case studies presented herein, the general guidelines for dealing with a gasoline leak (tank, pipeline, etc.) would be as follows:

GUIDELINES AFTER LEAK DETECTED

I. Preliminary Investigation

A. Field trip to assess site conditions.

B. Get good scale facilities map showing tank, substructures, etc.

C. Develop preliminary drilling program and lay out hole location on map. (Exploratory holes should be located at right angles to the tank, initially from 10-25 feet away.)

II. Geologic Research - Review geologic reports and maps.

III. Hydrologic Research

A. Groundwater Data

(1) Water Wells

(2) Observation Wells

(3) Groundwater Maps and Groundwater Gradient

B. Groundwater Reports

IV. Field Investigation

A. Holes - Final location and depth of holes will be determined by available drill sites, depth of groundwater and degree of contamination.

B. Depending upon the amount of gasoline and depth of the groundwater, additional holes should be drilling radiating out from the tank. The outer hole should be free of gas and define the outer limits of the contamination.

C. Holes should be drilled with hollow stem auger and samples obtained at 5-foot intervals. The holes should be logged and sampled, samples sniffed for gas fumes, put in a airtight glass sample jar under the supervision of a Registered Engineering Geologist. Sample labels should have location, hole number, depth, and date. Send samples to water quality lab for analysis.

(Note: Hole log should note moisture and free water if encountered.)

D. Holes should be drilled until clear of gasoline or groundwater is encountered.

E. Monitoring Well Casing 4-inch PVC lower 20± feet perforated should be installed, fill annular space with gravel to within 10 feet of the surface. Upper 10 feet should be filled with concrete or grout to prevent surface water from entering the well. Use stove pipe casing for upper 3 feet, top of casing a couple inches above ground surface, pour concrete past around casing with the outer edge tapering to the ground surface. This drains the surface water away from the well. Place gate cap over 4-inch PVC.

F. Have Monitoring Wells surveyed as to location and elevation.

G. Measured depth to water and gasoline using M-Scope (electrical conductivity measuring device) and tape with whistle attached, respectively.

H. Draw up cross-sections showing monitoring wells, depth to gasoline and water

V. Evaluated Results - (based on above)

VI. Recommend a Clean-Up Procedure

EPICHLOROHYDRIN IN SECONDARY CONTAINMENT SYSTEMS

William E. Witherow

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Carlisle, Pennsylvania

This paper first defines the problem of leaking underground fuel tanks and the real and potential threats to ground water in the U.S. Next, legislation describing how the EPA will regulate fuel tanks is presented. More importantly, several states have taken the regulatory lead on underground tanks. The legal and liability ramifications of Federal or state legislations and their effect on tank owners is briefly covered. Finally, leak detection and spill containment and collection system options such as secondary containment systems are included as possible solutions to this serious and emerging problem.

Finally, this paper specifically investigates a wide spectrum of physical properties especially aromatic hydrocarbon resistance for an unsupported epichlorohydrin lining that is being used today in several secondary containment systems in the United States. Laboratory testing was performed in accordance with the latest American Society of Testing and Materials (ASTM) standard test methods. Some of the more important physical properties are tensile strength, elongation, modulus, % swell (by volume) and water vapor permeability. Short- and long-term immersion studies were conducted in crude oil, diesel fuel #2 and a brine water/oil mixture at room and elevated temperatures. Also, comparative studies were performed to evaluate other materials such as thermoplastic chlorinated polyethylene (CPE) and Hypalon® and polychloroprene or neoprene (CR) rubber in ASTM oil #3, ASTM fuel C and American and European gasohol at room and elevated temperatures. A case history about the first epichlorohydrin secondary containment system in Ft. Lauderdale, FL showing typical installation and advantages such as cost effectiveness, watertightness, low permeability and flexible are presented.

Ground water contamination - a growing environmental issue in the '80's!

- A major oil company buys 23 homes at 150% of their market value and makes liability settlements with 27 families in East Meadow, NY.
  Total cost: $5 - $10 million.
  Cause: Leaking underground storage tanks.

- Another major oil company purchases 41 homes at 220% of their appraised value in Northglenn, CO.
  Total cost: $10 million.
  Cause: Leaking underground storage tank.
- A major oil company provides bottled water and filters for 23 homes in Dover and Walpole, MA.
  Cause: Leaking underground storage tank.
- Two major oil companies are installing a new water line to 37 homes and must restore the aquifer in Richmond, RI.
  Total cost: $1,165,500 to date.
  Cause: Leaking underground storage tanks.

The Problem

Although the volume of ground water is 50 times that of surface supplies, only ½ of it is suitable for potable use. Furthermore, approximately ½ or 116 million people in the U.S. drink their water from aquifers - an astounding consumption rate of 89 billion gallons per day! The situation becomes more critical since ground water contamination poses the greatest risk in heavily populated areas where ground water usage is heavy as in sole source aquifers. One gallon of gasoline can pollute 3/4 of a million gallons of water to yield a concentration of 1 part per million which is easily detected by the average person.

Estimates of the number of underground tanks in the U.S. and the number of tanks that are leaking vary in the industry. For example, sources state that there are between 1-4 million tanks and that 75,000-100,000 of them are leaking. The most common estimates range from 1.4-2.5 million tanks with 100,000 of them leaking. As was stated to the Senate Committee on Environment and Public Works, "neither the states nor the petroleum industry knows how many gasoline tanks there are in the country...or how many are leaking".

Individual state estimates are equally shocking. For example, New York estimates it has between 83,000-98,000 tanks; 16,000 or approximately 20% of them leak. In Maine, approximately 25% of its minimum 10,000 tanks are leaking. Michigan found 21% of the reported leaks were from gasoline tanks and that between 1977-79 35% of the reported leaks again involved gasoline tanks!

Unfortunately, the storage of gasoline products is not covered by the Toxic Substances Control Act (TSCA), the Resource Conservation and Recovery Act (RCRA) or the Comprehensive Environment Response, Compensation and Liability Act (CERCLA). However, based on pending legislation the EPA will probably regulate underground storage tanks under TSCA's Section 6, the storage of hazardous wastes under RCRA's Subpart J and gasoline storage and funds for cleanup of leaking gasoline tanks under CERCLA sometime this year. These laws would take effect within 12 months upon passage of this legislation.

Regulatory Response

Although ground water protection was traditionally the states' responsibility, there has been some recent legislation submitted to Congress to regulate ground water protection. For example, earlier Rep. Donald Ritter (R-PA) cosponsored HR 4985 which would amend Superfund by providing for the removal of gasoline from leaking tanks and fine violators $50,000/day or 2 years in jail for such infractions. In May 1984, Sen. David F. Durenberger (R-MN), the Chairman of the Senate Environmental and Public Works' Subcommittee on Toxic Substances and Environmental Oversight, sponsored S2649 which establishes the maximum contaminant level goals and the promulgation of national primary drinking water regulations for specific organic chemicals and contaminants. At the end of June 1984, Reps. Edward Madigan (R-IL) and Dennis Eckart (D-OH) cosponsored HR 5959 which although similar to S2649 would amend the Safe Drinking Water Act and also mandate that the states prepare and submit ground water protection plans to EPA within 30 months. Furthermore, no underground injection of hazardous wastes into or above aquifers within ¼ mile of a drinking water source would be allowed.

In January 1984, the EPA proposed a four-pronged ground water protection strategy. First, it would strengthen state ground water programs by setting aside funds to support state program development such as state registration of all tanks and a Chemical Advisory to assist service station personnel in identifying leak problems. Second, it would be able to cope with currently unaddressed ground water problems via surveying unaddressed threats and moving to regulate underground storage tanks by coordinating with such groups as UL, Steel Tank Institute, ASTM, American Petroleum Institute, etc. Third, the EPA would create a policy framework for guiding its program by establishing guidelines for consistency such as a uniform aquifer classification system. Finally, EPA would strenghten its own ground water organization by establishing an Office of Ground Water Protection, communicating with state program directors and instituting a committee consisting of Assistant Administrators and representative Regional Administrators.

The States' regulatory response will be discussed in depth in the next section.

Liability and Legislative Ramifications

Several insurance companies have initiated a slow leak insurance policy (a slow leak being defined as 1/100 gallon per hour) in response to some of this new legislation. For example, this insurance would help prevent slow leak environmental contamination since a tank owner must assume financial responsibility for any fuel leak contamination and would be required to have such mandatory insurance if he could not pay the out-of-pocket damages. The minimum coverage is typically $1 million at a premium ranging from $1000-1500 per $ million of coverage.

About 19 states and over 65 localities have promulgated laws governing tank performance. Tank manufacturers, oil marketers and API members have supported states as CA, MD, MI, FL (Chapter 17-61), CT, KS, OH, WI and IL which have passed such legislation while NY is presently receiving formal comments on its draft proposal. Localities prescribing more stringent requirements than the states' include Suffolk and Nassau Counties, NY, Barnstable County, MA, Santa Clara, CA, Dade and Broward Counties, FL, Cape Cod, MA Planning and Economic Development Commission, and Dallas/Ft. Worth, Austin and Houston, TX. Although Dade County, FL is considered to have the most prominent program, CA has over 60 communities with local programs. The L.A. region of the CA Regulatory Water Quality Control Board also has tentative regulations.

Design Options

Design options to solve the leaking tank problem can be delineated by the five basic components of an underground storage system. First, the corrosion of a storage tank can be prevented by use of a fiberglass, steel with cathodic protection, fiberglass-coated steel or a double wall tank. Second, the transfer facility can be properly planned and designed by use of good bedding material and backfill and anchoring equipment. Third, an overfill prevention system should include a level sensing and indicating device, high level alarm, automatic shutoff, interlock system and bypass prevention. Fourth, a leak monitoring system such as daily inventory control, line leak detectors, observation wells, automatic inventory control devices and tank tightness testing would increase leak detection. Finally, a spill or leak containment and collection system such as secondary containment has become a requirement in several states such as FL, KS, IL and CA.

Epichlorohydrin in Secondary Containment Systems

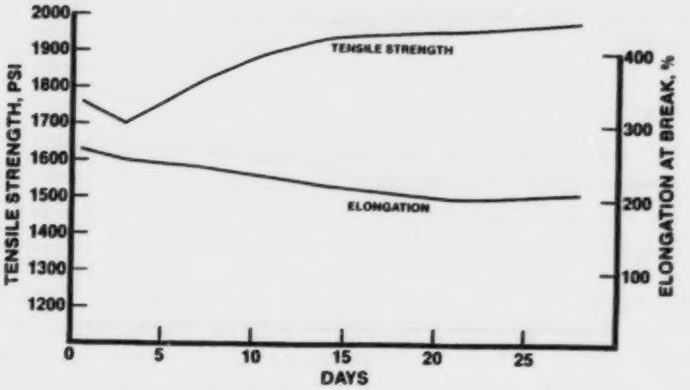
The selected barrier should not allow the leaked product to escape into the ground water. Based on generally accepted criteria, the following factors should be considered when selecting a secondary containment liner (Woods 1984 and Garver 1983):

1. Type of material being stored - includes compatability as well as material integrity and impermeability when exposed to the stored liquid.
2. Local environmental conditions - near or above (sole source) aquifer, depth of water table, etc. needs to be determined.
3. Legislative requirements - local ordinance more specific or stringent than state law?

Epichlorohydrin elastomers which were first synthesized in the 1960's were primarily used in demanding automotive applications such as fuel hose and tubing, seals, gaskets, etc. Their inherent aromatic, solvent and oil resistance together with the cross-linked state of the finished membrane provided numerous features that several major oil companies felt would contribute to reliable service in a secondary containment system:

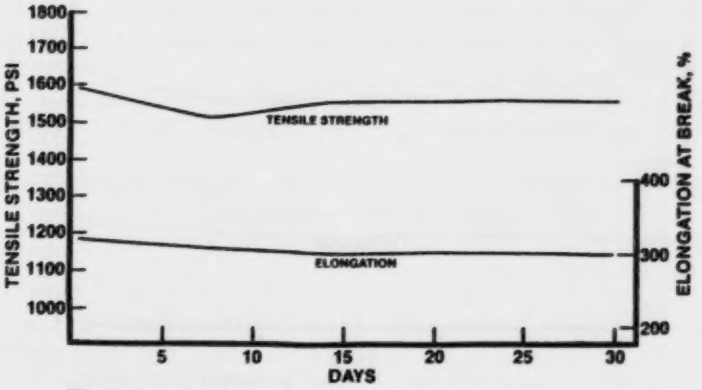
- Durable and flexible to effectively absorb shifting soil conditions.
- Exceptional oil, fuel and solvent resistance.
- Maintain material integrity with aromatics.
- Cost effective.

The second and third features were substantiated by over 11 years of immersion data. For example, epichlorohydrin reaches dimensional stability in crude oil at 212°F in 15 days (Figure 1), diesel fuel #2 at room temperature in 14 days (Figure 2) and fresh water at room temperature in 9 days (Figure 3).



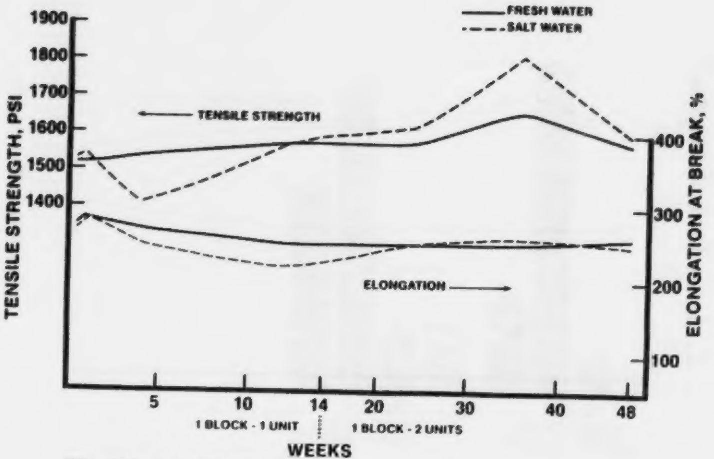
NOTE: Herclor H = Epichlorohydrin CO

Figure 1
IMMERSION IN CRUDE OIL
HERCLOR H - 212°F EXPOSURE



NOTE: Herclor H = Epichlorohydrin CO

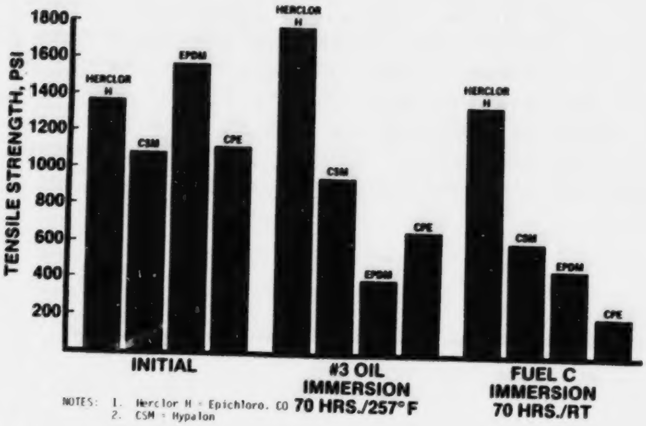
Figure 2
IMMERSION IN DIESEL FUEL #2
HERCLOR H - RT EXPOSURE



NOTE: Herclor H = Epichlorohydrin CO

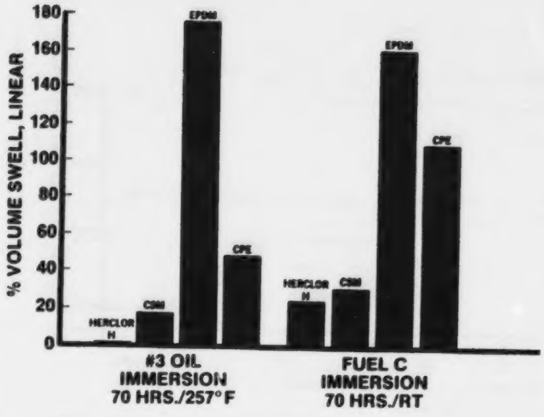
Figure 3
WATER IMMERSION
HERCLOR H - RT EXPOSURE

Next, epichlorohydrin was evaluated against other similar membranes. Basically, it showed the highest retention of tensile strength (Figure 4) and the lowest % swell by volume (Figure 5).



NOTES: 1. Herclor H = Epichloro. CO
2. CSM = Hypalon

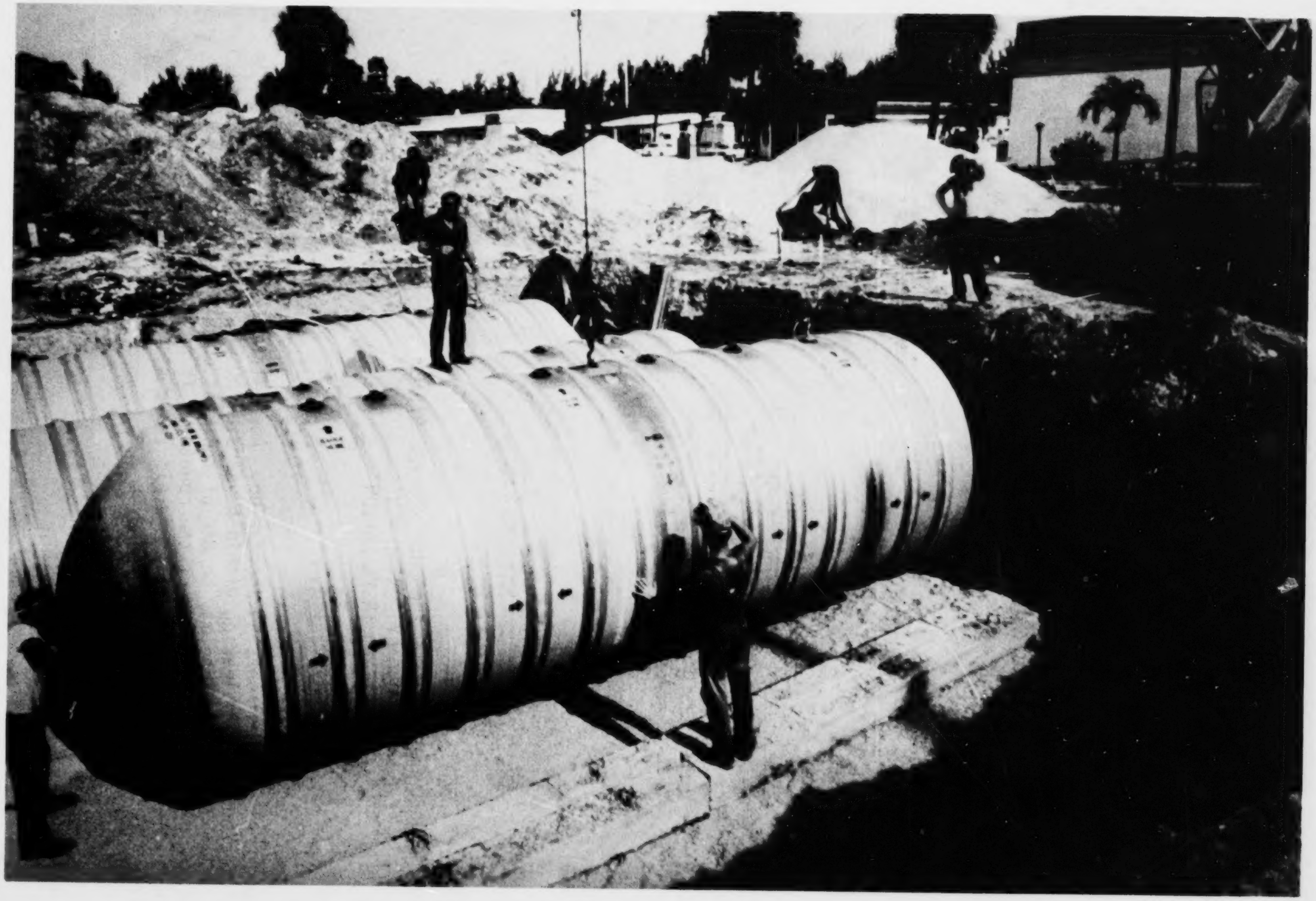
Figure 4
COMPARATIVE STUDY



NOTES: 1. Herclor H = Epichlorohydrin CO
2. CSM = Hypalon

Figure 5
COMPARATIVE STUDY

The installation of our first secondary containment liner occurred at a new major service station in Ft. Lauderdale, FL in May 1983. After the "dry" pit was dug with a backhoe, the liner was lowered by crane into the ground where it conformed to the bottom and sides of the pit. Some pea gravel was backfilled and then four fiberglass fuel tanks, each capable of storing from 8,000-12,000 gallons, were lowered into the pit. 12" x 12" reinforced concrete deadmen were placed at the sides of these tanks (Figure 6) and attached to the fiberglass straps which would cover the tanks. Trenches which contain the product piping connecting the tanks to the pumps were also lined. Then the remaining pea gravel was backfilled as the syphon,observation well (Figure 7) and product pump were installed. Next, water is added to the pit so that the water level is a minimum of 6" above the tank tops. Finally, a 8" thick reinforced concrete slab is placed over the tankfield. A typical installation view is shown in Figure 8.



New Epichlorohydrin Liner from Carlisle SynTec Systems resists hydrocarbons offering protection in a variety of applications including secondary containment in underground fuel storage.

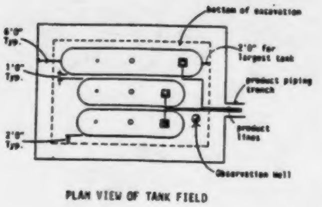
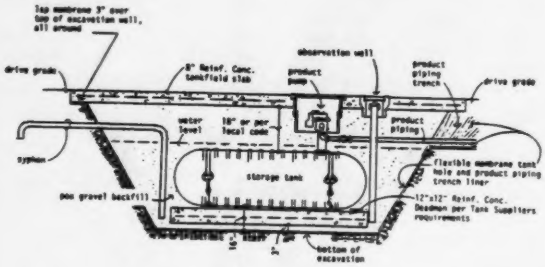

bottom of excavation
6'0" Typ.
1'0" Typ.
2'0" Typ.
2'0" for largest tank
product piping trench
product lines
Observation well
PLAN VIEW OF TANK FIELD

Figure 7
OBSERVATION WELL PLACEMENT IN SECONDARY CONTAINMENT SYSTEM


lap membrane 3" over top of excavation wall, all around
drive grade
8" Reinf. Conc. tankfield slab
product pump
observation well
product piping trench
drive grade
water level
18" or per local code
product piping
syphon
storage tank
flexible membrane tank hole and product piping trench liner
pea gravel backfill
12"x12" Reinf. Conc. Deadmen per Tank Suppliers requirements
bottom of excavation

NOTES:

1. Water level within lined tank hole to be maintained 6" above tank top.
2. All excavation backfill to be pea gravel.
3. Liner material should be lapped 3" over the top of the excavation wall, all around. The liner material must be temporarily "weighted" around the top of the excavation wall until the pea gravel backfill is completely in place.

Figure 8
UNDERGROUND PETROLEUM PRODUCT STORAGE SYSTEM
SECONDARY CONTAINMENT
TYPICAL INSTALLATION

Conslusion

The number of leaking tanks is increasing. An additional 350,000 tanks are expected to begin leaking within 5 years. The EPA reports that because of these leaks, gasoline may be one of the most common causes of ground water pollution in many parts of the country. If no action is taken in the next decade, up to 75% of these old (bare steel) tanks may begin leaking.

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Mr. Witherow graduated with a B.S. in Civil Engineering from VMI as a Distinguished Military Graduate. He served in the U.S. Army Corps of Engineers, has worked as a project engineer with a consulting firm and was a director of public works. He is employed by Carlisle SynTec Systems as product manager and will receive his MBA in 1985.

Mr. Witherow has coauthored several technical papers which have been presented at various international symposia, conferences and universities. He is actively involved on several committees in ASTM, NSF and ASCE and also is a member of AWWA, WPCF, CSI and WWEMA.

Session II
Dealing with Injection Wells

AN IN-DEPTH SURVEY AND ASSESSMENT OF DEEP INJECTION WELLS USED TO DISPOSE OF HAZARDOUS WASTE - PRELIMINARY REPORT

Victor J. Kimm
Francoise M. Brasier
Mario Salazar

Office of Drinking Water
U.S. Environmental Protection Agency
Washington D C

In 1981, the Office of Solid Waste of the U.S. Environmental Protection Agency (EPA) conducted a survey of hazardous wastes management practices by sending questionnaires to owners and operators of facilities who had notified the Agency that they handled hazardous wastes, pursuant to notification requirements under the Resource Conservation and Recovery Act (RCRA). This survey identified 73 hazardous waste injection facilities used to dispose of an estimated 7.3 billion gallons of waste in that year. At the same time, in early 1983, several bills were introduced in Congress that would have mandated an EPA study of all wells used to dispose of hazardous waste, and questions were raised about the extent and safety of the practice.

In the Summer of 1983, the Office of Drinking Water responded to these concerns by conducting a survey and assessment of hazardous waste injection practices. In order to obtain a comprehensive inventory of all wells used to inject hazardous wastes, Regional EPA offices were provided with a questionnaire, which was either transmitted to the states or answered by the Regional staff with the help of a contractor. At the same time, in August and September of 1983, personnel from EPA headquarters, Regions and States jointly visited 20 hazardous waste disposal facilities, believed to include hazardous waste injection wells and chosen to represent a cross-section of geographic areas, wastes characteristics and types of operations. Hazardous wastes were, in fact, injected at only 17 of these facilities, which accounted for a total of 53 wells. The waste streams injected at the other three facilities were not hazardous.

The questionnaire devised by ODW listed data elements regarding owner/operator identification, operational status, geology of injection and confining formations, hydrogeology, well design and construction, operation, well testing and monitoring, inspection and surveillance, permit status and limitations. This questionnaire was filled out on site and the information verified to the greatest extent possible for the 17 facilities where site visits were made. For all other wells the questionnaires were filled using data available in State files.

The results of the study, which are now being analyzed and verified will be published, early next year. This paper discusses some of the preliminary findings as they pertain to the number, age, distribution, and general characteristics of injection wells used to inject hazardous wastes. It also briefly reviews the regulatory framework under which hazardous waste injection wells are operating and ODW's position on injection wells.

Results of the Survey

General Information

Based on the information available at this time, we believe that there were, as of October 1983, 188 active Class I hazardous waste injection wells (Class I HW wells) distributed among 102 facilities. Active wells are those that received waste at some time during 1983, either on a continuous or intermittent basis as well as stand-by or back-up wells in operational condition.

Class I wells are defined in the Agency's Underground Injection Control (UIC) regulations, as wells used to inject wastes below all underground sources of drinking water, i.e. all aquifers in which the water contains less than 10,000 mg/l total dissolved solids. Class I wells represent only a small proportion of the wells used to dispose of waste. The vast majority of disposal wells are in fact used to dispose of oil field brines. EPA's inventory shows that approximately 24,000 of these wells are currently active, more than 5,000 of them in Texas alone. In contrast there are only 649 Class I wells in the inventory and approximately 71% of these wells are used to inject wastes that are not defined as hazardous under RCRA.

More than 90% of these wells are "on-site" wells that is they are owned and operated by the waste generator. Only 10% of the wells are commercially operated facilities that collect a service fee for the disposal of a variety of wastes. These are usually referred to as "off-site" wells, and active ones are located in Louisiana, Ohio, Oklahoma and Texas.

Wells used to inject hazardous wastes are concentrated in a few areas of the country (Figure 1). A vast majority of the wells are located along the Gulf Coast and near the Great Lakes. Louisiana and Texas alone account for 62% of the wells. Other States with sizeable numbers of HW wells are Michigan, Indiana, Ohio, Illinois and Oklahoma. In general, the wells are located in areas of oil and gas production where data on deep formations are readily available.

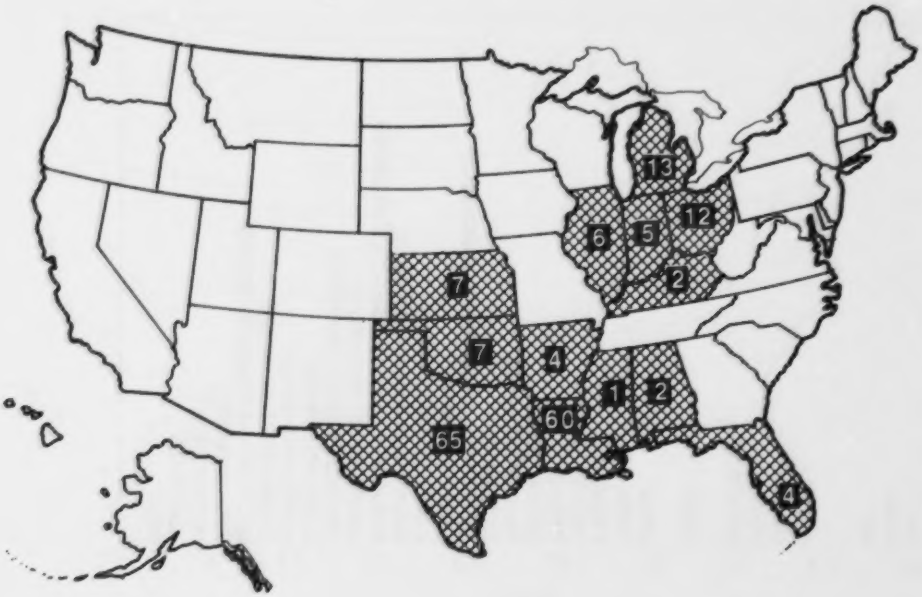

Fig. 1. Distribution of Class I HW Injection Wells

The majority of the wells active today were drilled in the mid-1960s to the mid-1970s (Figure 2). There was a peak in start-up of injection wells in 1973, 1974, and 1975, probably as a result of the implementation of the Clean Water Act which established stringent pollution control requirements for discharges to surface waters. The enactment of RCRA, on the other hand does not seem to have had an effect so far on the well population. This may be due to the fact that for approximately two years, between the effective date of the RCRA regulations (August, 1980) and the time the first UIC program in a State was approved, (January, 1982) HW injection wells were in a regulatory limbo, with construction of new wells prohibited under RCRA (because no HW facilities could be constructed without a federally-approved permit) and the permitting requirements devised under UIC not in force until State programs were specifically approved 1/. The fact that some wells were constructed during that period is probably due to the lag time between permit issuance and start of construction.

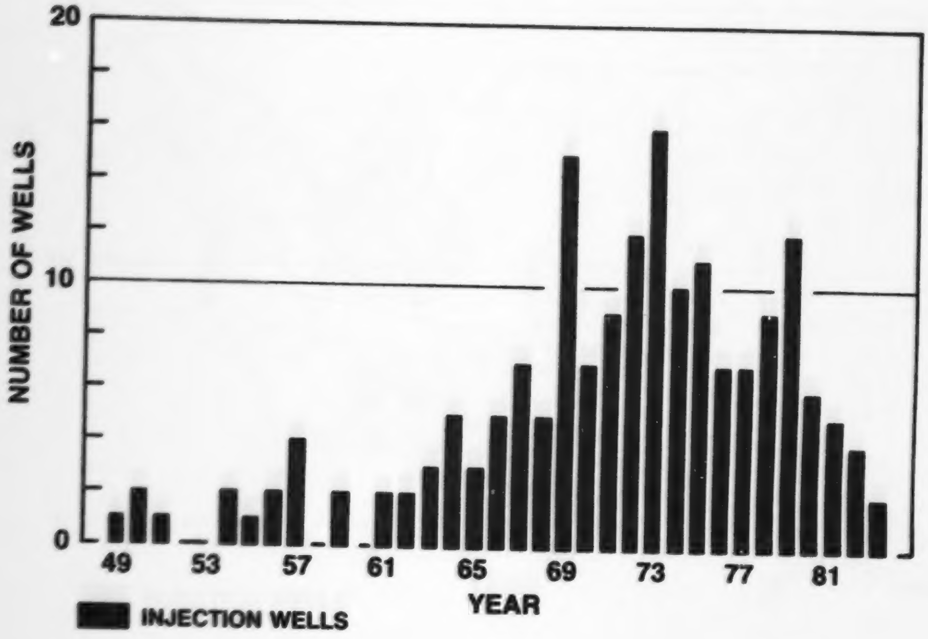

Fig. 2. Age Distribution of Class I HW Injection Wells

Verified data available on 124 Class I HW wells show that 7.54 billion gallons of waste were injected in 1983 in those wells. In addition, verified average injection rate data for 43 wells would account for an additional 3.36 billion gallons of waste assuming constant injection.

1/ EPA did promulgate under the authority of RCRA, 40CFR Part 267 which would have allowed Regional Administrators to issue temporary RCRA permits to injection wells. These provisions were never used.

Available data, however, are not sufficient to calculate the actual contribution of hazardous waste to the total injected volumes. Also data are sketchy as far as concentration of the waste streams, so that the hazardous waste loading to the subsurface from HW injection wells cannot be estimated at this time. The Office of Drinking Water is currently gathering more data on this subject and the final report will provide better information.

The biggest user of HW Class I wells is the chemical industry (Figure 3). Manufacturers of organic chemicals account for 43% of the injected wastes, while the petroleum refining and petrochemical industries account for 20% of injected volume. Twenty-eight percent of the injected wastes are generated by other chemical manufacturers. Only four percent of the total volume of injected waste is handled by commercial (off-site) waste disposers.

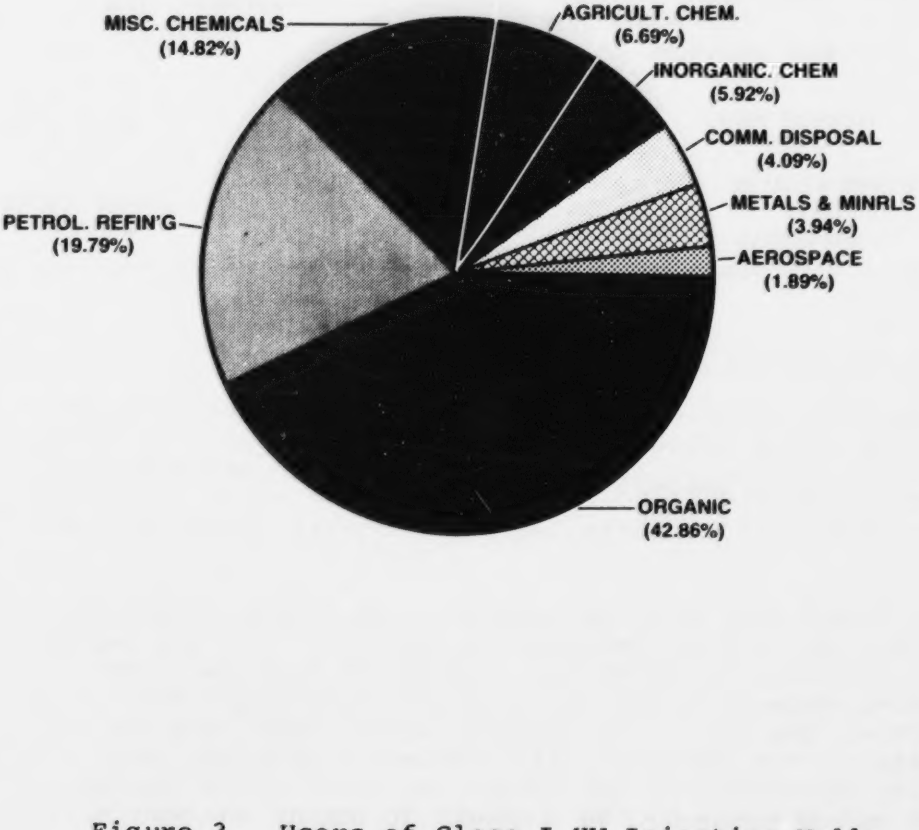

Figure 3. Users of Class I HW Injection Wells

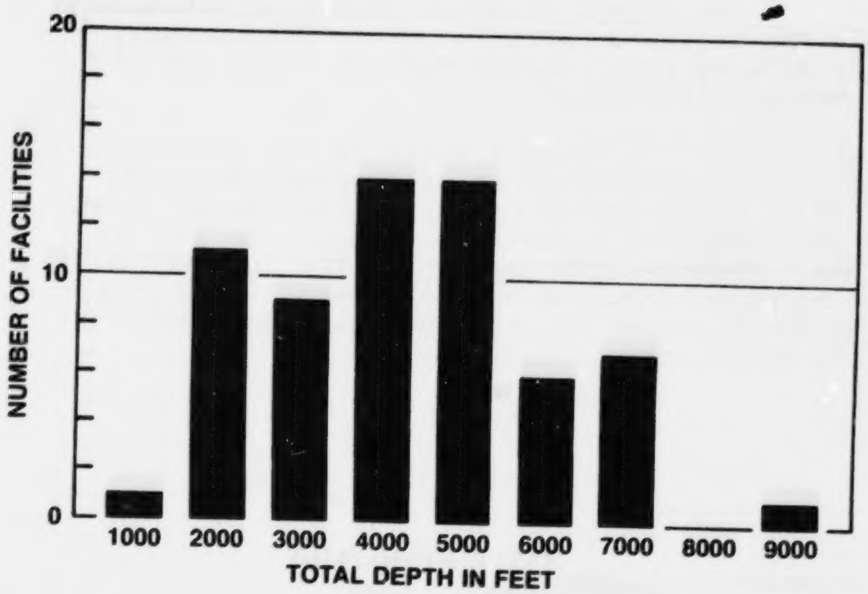

Figure 4. Depth of Class I HW Injection Wells

Well design and construction

The inventory revealed that Class I HW wells are in fact, deep injection wells (Figure 4). With a few exceptions, these wells are completed below 2,000 ft. The average depth of all wells is approximately 4,000 feet. The deeper wells are found in Texas and Mississippi where the depth usually exceeds 4,500 feet.

The study found that most HW injection wells are constructed with redundant protective features.(Figure 5). All the wells are constructed with at least two strings of casings. The surface casing extends below the base of 10,000 mg/l TDS water in 57% of the wells, is usually carbon steel and is cemented back to the surface. All the wells also had long string casing extending to the injection zone. This casing is cemented all the way to the surface in 74% of the wells.

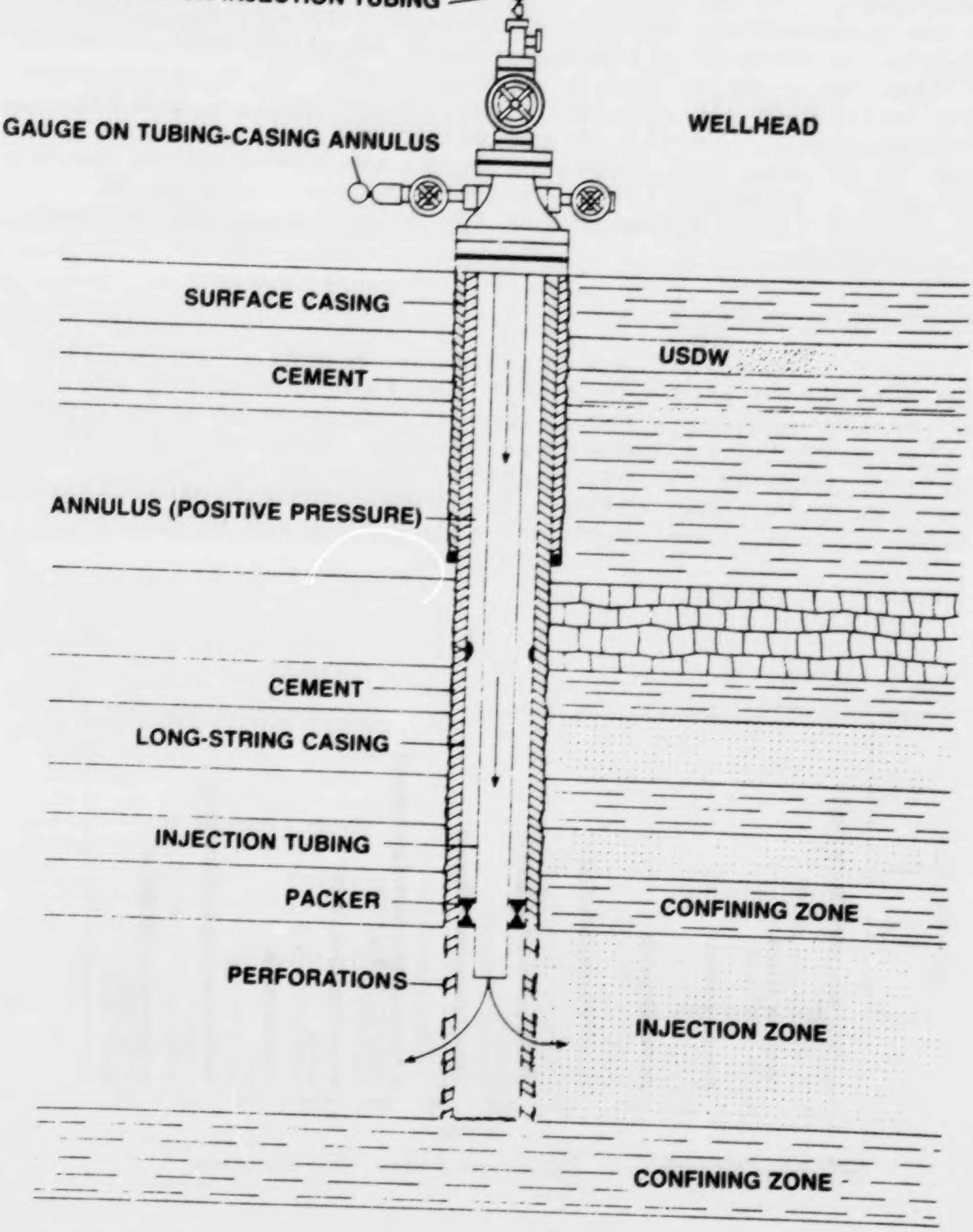
GAUGE ON INJECTION TUBING
GAUGE ON TUBING-CASING ANNULUS
WELLHEAD
SURFACE CASING
USDW
CEMENT
ANNULUS (POSITIVE PRESSURE)
CEMENT
LONG-STRING CASING
INJECTION TUBING
PACKER
CONFINING ZONE
PERFORATIONS
INJECTION ZONE
CONFINING ZONE

Figure 5. Typical Class I HW Well

It is usually carbon steel although other materials such as stainless steel, special alloys and figerglass were also encountered. In addition, thirty five percent of the wells have an intermediate string of casings. In all cases injection is through a tubing. The typical injection tubing is 5.5in. in diameter and is carbon steel. Thirteen percent of the wells had fiberglass tubing, 10% fibercast and 5% stainless steel. In 93% of the wells the tubing is set on a packer at or near the injection zone, the other wells use a fluid seal. It would take a simultaneous failure of at least two of these elements for the injected wastes to reach the environment.

Hydrogeologic setting

A vast majority of the Class I HW injection wells (71%) are completed in sand and sandstones formations, 15% are completed in carbonate formations (limestones or dolomites) and the remainder in shaley sandstones. Most of the confining zones are composed of shales (66%), followed by shaley sandstones (14%), and shaley limestones (12%). Other examples of reported confining zones are silt, clay and dolomites.

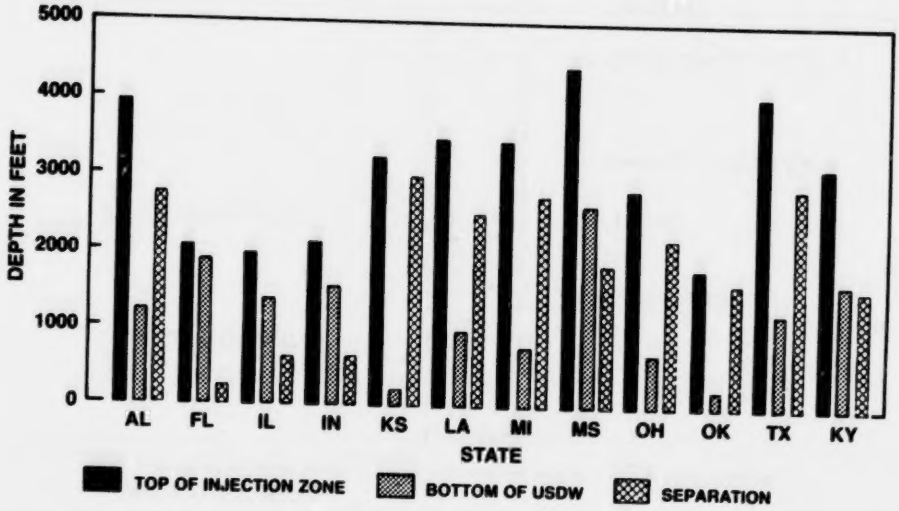

Figure 6. Average Depths by State

In the Great Lakes Area, the disposal zone is usually the Mount Simon formation, a 600 to 700 foot thick sandstone found at approximately 3,000 feet of depth. Confining beds of limestone, dolomite and siltstone approximately 1,300 foot thick separate the Mount Simon from the base of 10,000 mg/l TDS water. In Texas and Louisiana, the injection zones are typically unconsolidated sediments of tertiary age and are more than 4,000 feet deep. They are separated from the base of fresh water by one to two thousand feet of sediments that include several silt and clay intervals.

The data show that in most instances there is good separation between the injection zone and the base of 10,000 mg/l TDS water (Figure 6). In more than fifty percent of the cases this distance is more than 2,500 feet. There is of course greater separation from the base of 3,000 mg/l TDS water, the outer limit of water usually considered useable as a source of drinking water. This distance is greater than 2,500 feet in approximately 70% of the wells in the inventory.

Well operation

Even though the questionnaire contained elements for chemical composition and concentration of injected waste, the data in State files were insufficient to characterize the types of wastes injected in HW Class I wells. Usually, mixtures of diversified individual wastes streams are mixed in holding ponds or tanks prior to injection. Better data were obtained during the site visits. Approximately 70% of the injected streams at these facilities were acidic, with hydrochloric acid the most common component. Generally, wastes streams are treated and blended prior to injection. The most common forms of treatment were sedimentation, disinfection, filtration, oil and grease removal, neutralization and dilution.

All injection wells have a limitation on the injection pressure. This limitation is in all cases set below the fracture pressure of the receiving formation, and is usually calculated based on a hydraulic fracture gradient. Twenty percent of the wells in the inventory injected waste by gravity flow.

All operators are required to monitor injection pressure, flow rate and cumulative volume in most cases using continuous monitoring devices. In approximately 90% of the wells the tubing-casing annulus is filled with fluid upon which positive pressure is applied. The pressure is monitored continuously to detect leaks in tubing casing or the packer. The majority of the assessed facilities had an automatic shut-off system that is activated whenever one of the monitored parameters reaches a given level.

The data show that wells are extensively tested prior to being put in operation in order to ascertain the mechanical integrity of the well. The integrity of the tubular goods was tested by pressure tests, caliper logs or radioactive tracer surveys. In all the sites visited a log such as cement bond, velocity or temperature had been run to confirm the soundness of the cementing job. The review of State files revealed that similar tests had been run in most of the wells in the inventory. In addition, all but a few of the assessed facilities have implemented corrosion control methods in order to preserve the integrity of the well materials. These include use of corrosion resistant material and addition of corrosion-inhibiting fluid to the annulus. Two of the facilities visited have installed systems to expose coupons of well materials to the waste stream. These coupons are weight-tested at regular intervals to give early warning of corrosion. The UIC regulations also require periodic testing of mechanical integrity during the life of the well. This practice is already in place in most of the facilities assessed with some facilities running yearly tests.

Regulatory Framework

Class I HW injection wells, like all other injection wells are subject to the requirements of the UIC regulations promulgated under the Safe Drinking Water Act, and administered either by States (primacy States) or by EPA. Prior to implementation of of the federal regulations, all the wells were subject to State regulations. When a UIC program is promulgated or adopted in a State all existing class I wells must be repermitted, in accordance with the Federal requirements. Most of the Class I HW wells are located in States that have received primacy. In those States owners or operators of existing wells must submit a permit application within 4 years of program approval. Two of these States, Texas and Louisiana, which account for the majority of Class I HW wells were among the first to receive primacy (Texas in January 1982, and Louisiana in April 1982) and have issued permits that meet or exceed all Federal requirements to 41 wells. Most other States have put Class I HW wells at the top of their priority list an will review existing wells within the first two years of the program.

EPA has direct jurisdiction over 13 facilities in Michigan, Indiana, and Kentucky for a total of 21 existing wells. In these States, the regulations require that owners or operators of existing wells submit permit applications during the first year of the program and plans are to review these applications on an expedited schedule.

Conclusions

The practice of using injection wells to dispose of waste started in the oil fields in the 30s and is considered an environmentally desirable method for disposing of oil field brines and other waste fluids resulting from oil and gas production. Injection of industrial waste started in the late fifties and so far has proven a reliable method for disposing of certain wastes. Recent studies by the National Research Council, the General Accounting Office and the Office of Technology and Assessment have all cited waste injection as an inherently better technology than other methods of land disposal of wastes. Only one Class I well is associated with a site on the National Priority List of Superfund sites. Injection at this particular site stopped in the early 70s and examination of the data indicates that the well would not have been in compliance with the current UIC requirements. There have been other problems associated with injection wells, but none that have resulted in documented cases of drinking water contamination. All of the problems occurred before the UIC regulations were in place and could have been avoided if the UIC requirements had been met. They can be tied to improper siting, excessive injection pressures, and lack of monitoring for mechanical integrity.

As we review the results of the Study and gather more information on how injection wells are currently operating, we may generate additional guidance and if appropriate added regulatory requirements. The assessment has shown us that some operators and States already go beyond the minimum standards of the UIC regulations and we will want to ensure that best technology is applied to all HW injection wells.

What we have seen so far leads us to believe that while injection wells are not a panacea, when properly sited, operated and monitored, and for the type of wastes that lend themselves to the technology, they can be an environmentally sound method of disposal. However, whether a specific well should be allowed to operate should remain a case-by-case determination since the interaction between the hydrogeologic setting, construction material and type of waste injected must always be considered. In addition, congressional action related to the reauthorization of RCRA 1/ may force us to reexamine the issue of hazardous waste injection as part of the potential ban of one or more methods of land disposal for specific hazardous wastes.

Amendments to RCRA were enacted on November 9, 1984.

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MR. KIMM IS AN ENGINEER BY TRAINING, HAVING RECEIVED BACHELOR'S AND MASTER'S DEGREES IN SANITARY ENGINEERING FROM MANHATTAN COLLEGE AND NEW YORK UNIVERSITY. HE ALSO SPENT AN ACADEMIC YEAR AT PRINCETON STUDYING ECONOMICS AND PUBLIC ADMINISTRATION AS A RECIPIENT OF A NATIONAL INSTITUTE OF PUBLIC AFFAIRS FELLOWSHIP. HE IS A LICENSED PROFESSIONAL ENGINEER IN NEW YORK AND PENNSYLVANIA.

HIS PROFESSIONAL CAREER IS DIVIDED BETWEEN CONSULTING ENGINEERING, DEVELOPMENT WORK, PRIMARILY IN LATIN AMERICA, AND IN THE MANAGEMENT OF ENVIRONMENTAL PROGRAMS AT EPA.

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Mario Salazar-Bio 11/14/84

Mario Salazar was born in Columbia, S.A. and became a U.S. Citizen in 1980. He obtained a B.S. in Civil Engineering from the University of Maryland in 1972 and a M.S. in Environmental Engineering in 1981. Mr. Salazar has worked in the private sector as a consultant for five years, State government in various environmental programs for three years and has been in EPA since 1980. His responsibilities at EPA include managing the implementation of the UIC program in respect to the injection of hazardous waste and the development of guidance for quality assurance of data generated for the program.

PRACTICAL DISPOSAL WELL DESIGN FOR THE PREVENTION OF GROUND WATER CONTAMINATION

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Abstract

The design of a disposal well system requires the consideration of many factors, however, none are as important as the protection of our potable ground water. The principle factors effecting proper ground water protection are the casing design and obtaining a good primary cement job. The overwhelming factor in the success of a primary cement job is the efficiency with which the fluid system in the annular space to be cemented is replaced by the cement slurry. The efficiency is effected by many factors such as depth, temperature, pressure, well bore geometry, fluid properties, contact time, fluid density difference, pipe centralization, pipe movement and many others. Commercially available electric logging tools such as the Temperature Log, Radioactive Tracer Survey, Cement Bond Log and Cement Evaluation Tool can be used to substantiate proper ground water protection. The authors' practical experience and case histories show the successful use of these principles for practical disposal well design and the protection of our ground water supplies.

Practical Casing Design

The primary objective of a casing design is to provide a length of pipe that will withstand the forces encountered when used to line the internal surface of a drilled hole. The design should allow for the control of both expected and unanticipated well conditions. Since all conditions cannot be addressed, design criteria is controlled by reasonable economic and safety standards.

The size casing required is the first decision that must be made. Once this decision is made, the thickness of the casing wall and metallurgy can be determined. The wall thickness of tubulars, used in well systems, is normally specified to in pounds per foot of length and the grade usually refers to yield strength. Casing comes in various lengths and when run into the hole is joined together by welding or special threaded connections.

Well planning begins with the selection of a bit and casing program that will be determined by both well conditions and experience. The bit design is controlled by the type rock to be penetrated and the size casing that will be set and cemented. The casing design is based on the load conditions normally encountered, viz, collapse, burst, tension and compression.

The API publishes bulletins and specifications (API Bul. 5C2 1975, API Spec. 5A 1976 and API Bul. 5C3 1974) which contain minimum properties for various casings in burst, collapse and tension. In order to use API's monogram, the manufacturer of the casing must meet minimum standards as set by the API. Non-API casing normally manufactured by the same standards is usually satisfactory; however, in critical wells the designer should satisfy himself that all standards are met.

Collapse is the first condition considered in casing design and represents the casing external load capabilities (Figure 1). Since the greatest hydrostatic pressure occurs at the bottom of casing, we must design from the bottom up. Thus the weight and grade requirements may be reduced as the design moves nearer the surface to optimize the casing program. These lower requirements still maintain the desired level of safety but allow the use of a less expensive weight of pipe. Therefore, when designing for collapse, the lowest cost casing of sufficient strength to withstand the collapse load plus the appropriate safety factor, should be used. When a choice exists between the use of a lower grade/or heavier weight and a higher grade/or lower weight casing, each having the same cost, the lower weight/and higher grade casing should be used. This reduces both the load on the rig and the tension load requirements.

Burst load consideration in casing design are usually the reverse of the collapse loads; i.e., the most important part of the casing for burst design is the uppermost section. Burst loads are a result of well pressures, internal hydrostatic pressures, cementing pressures, fluid density differences, well stimulation and other overbalanced conditions between internal and external pressures (Figure 2). The burst resistance of casing is effected by tension; however, burst is almost always the controlling design factor.

Tension is the load imposed by the weight of the casing itself, i.e., each joint must support the weight of the joint below it (Figure 3). The greatest tensile load is at the surface and decreases to zero, or neutral at some point down the casing. Most drilled holes contain fluid and a neutal point occurs due to buoyancy. Above this neutral point, the casing is in tension and below it the casing is in compression.

After the design criteria has been established, based on collapse and burst calculations, the casing properties are checked to make sure they meet the tensile strength requirements. When these requirements can no longer be met, a change is made. The change may be in the type

FIGURE I
COLLAPSE FORCES

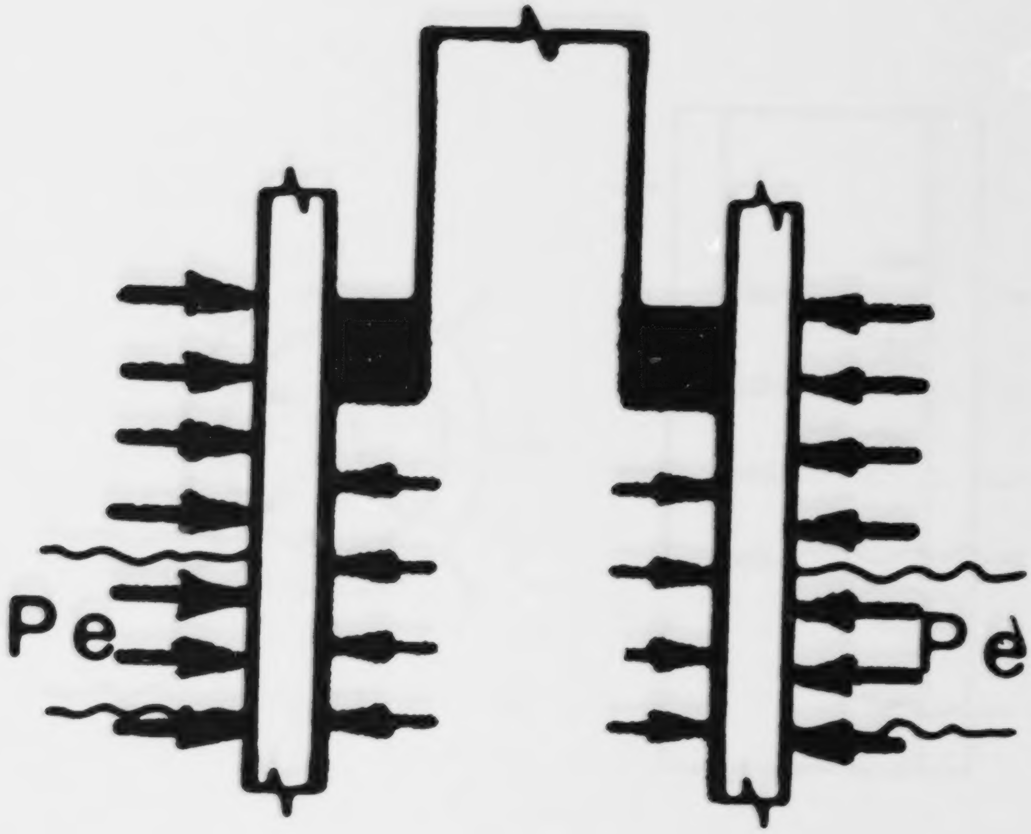
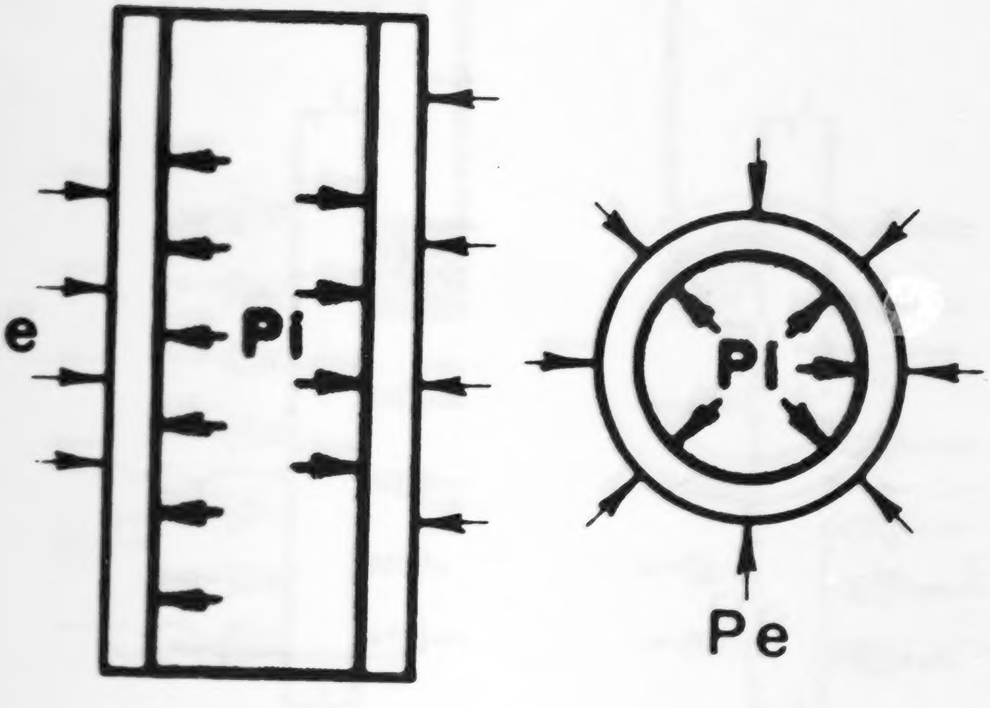


FIGURE 2
BURST FORCES



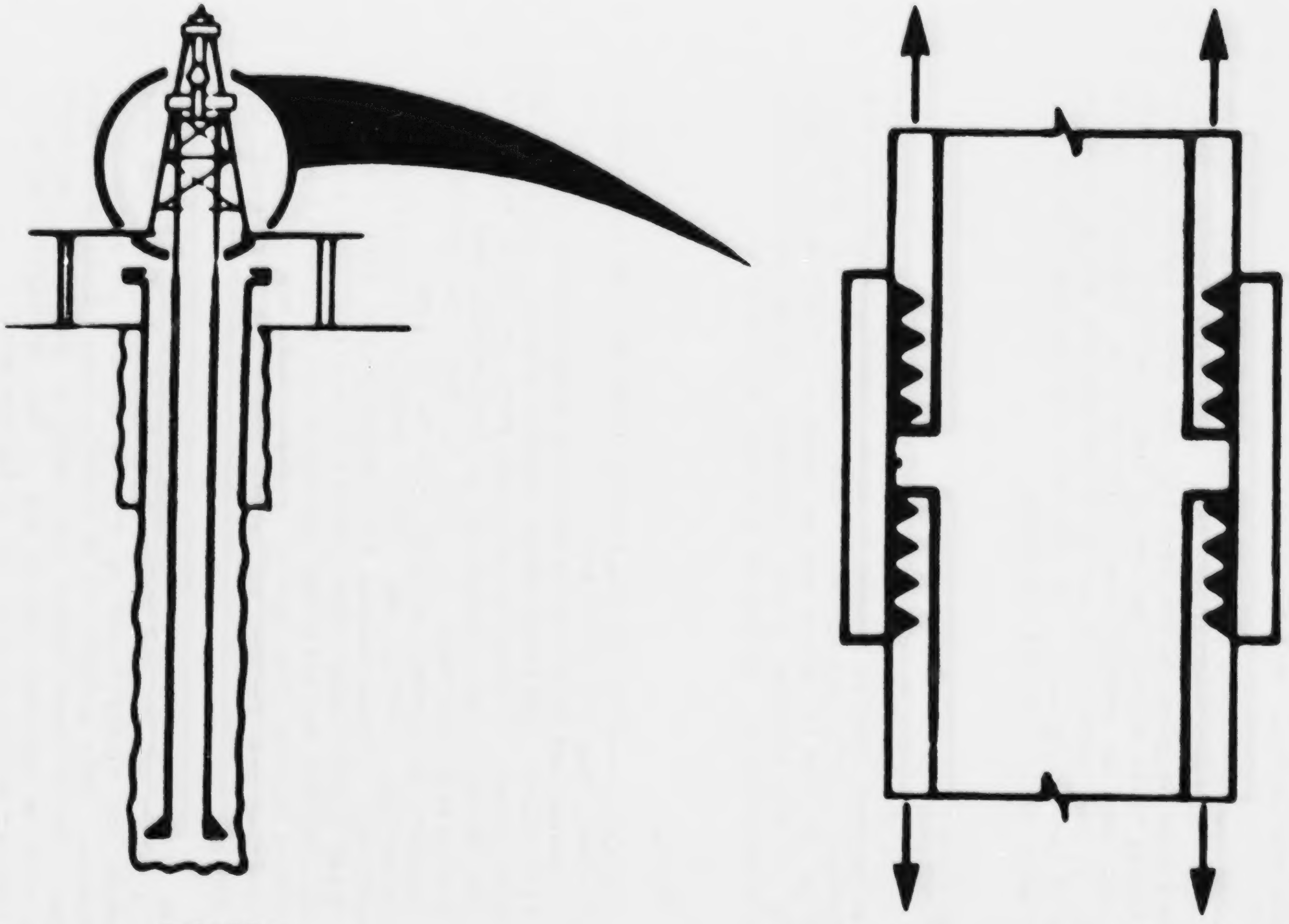


FIGURE 3
TENSION FORCES ON THREADS

WEIGHT OF STRING

of connection, the weight of casing or grade of steel; all changes should provide adequate strength for the least costs. Other considerations for tension design are the additional loads that may be placed on the casing in the event the casing becomes stuck during installation, during reciprocation, while cementing, or when supporting the casing from the wellhead assembly.

Compressive loading enters into the design consideration at the top portion of the surface casing or load supporting casing unit. Consideration should be given to the fact that the cement sheath outside the surface casing helps support the casing load, therefore, the surface casing does not need to support the entire load of itself or other casing units. To some extent, the design considerations change based on the location of the casing unit. In disposal well systems, the casing program is usually limited to the following:

1. Conductor Casing

Burst, collapse and tension are not considered in design of this pipe. The mechanical loading caused by drive hammers, supporting the blowout preventer (BOP) stack, etc. are difficult to quantify. Generally, pipe is selected which has at least a .3 to .4 inch wall thickness and of the cheapest grade available. Since metallurgical properties are not critical, welding on the pipe is permitted.

2. Surface Casing

Burst, collapse, tension and compression loads must be considered. Net burst conditions that can occur are:

- Increased mud weight while drilling below surface pipe.
- Well control operations where surface pressure results. The most severe of these is the case where no fluid exists in the wellbore thus eliminating the hydrostatic head. If the hydrostatic pressure is removed, then the bottom hole pressure and surface pressure will be the same.
- Cementing, testing and other routine applications of pressure. These values are generally the result of the final design rather than factors in design. Once a design is complete the cementing procedures are written to prevent overpressuring the casing. Also, BOP and casing test pressure are limited to levels below the rating of the pipe.

Collapse pressures on surface casing result primarily from the hydrostatic pressure of the mud column outside the casing. In some cases, pressure can slowly build up on the annulus as a result of thermal expansion. This pressure is an added collapse force which must be recognized. For collapse design the casing is generally considered to be empty with an external loading due to the hydrostatic pressure of the drilling mud weight in which the string was set.

Tension design for surface casing is rarely significant. Most surface strings are relatively short and tensile loads due to buoyed pipe weight are far below the rated yield. As discussed, this string is subjected to compressive loading since all additional strings landed in the wellhead assembly are usually attached to the surface casing. A cement horsecollar or cement circulated to the surface behind the surface casing will provide load carrying capacity. However, once all strings have been designed, the combined buoyed weight should be checked against the compressive strength of the top few joints. If compressive strengths are exceeded, a few joints of heavier pipe should be substituted.

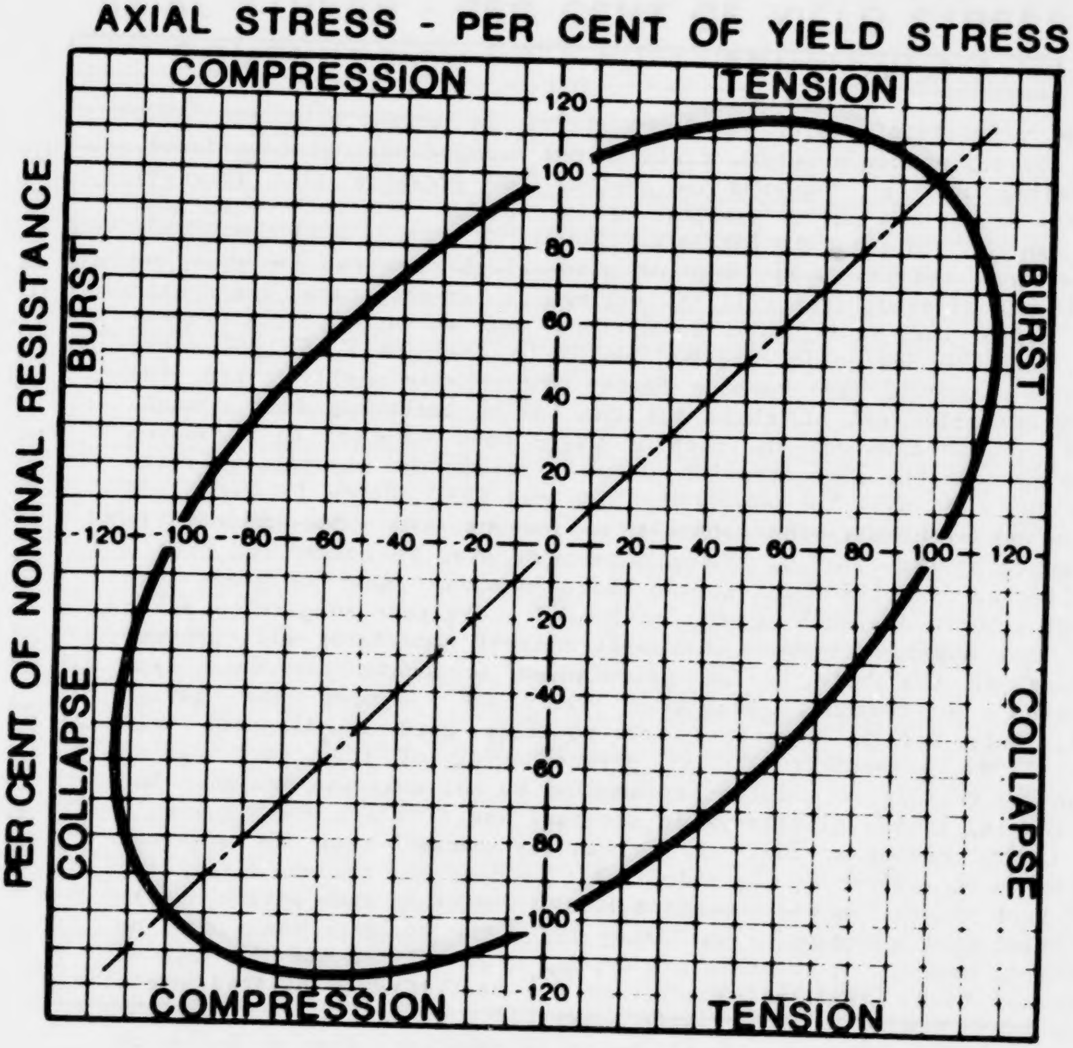
3. Protective Casing

The protective casing or long string is potentially the most complex to design since burst, collapse and tension must be considered. This casing unit is required to protect low pressure (low fracture gradient) formations and to isolate specific zones. Numerous conditions can be expected to generate forces on the casing. Because of the number of conditions and the wide range of assumptions that can be made, no specific set of conditions will fit all wells. In each case, the design should allow for anticipated conditions and an appropriate safety factor. Casing design in disposal wells is frequently not given the attention it should have because forces are usually small. Also, the well is normally full of fluid and the casing backed-up with cement. However, let us consider the case of waste acid disposal in carbonate reservoirs. If the concentration of acid exceeds the solubility limits of the CO_2 that will be generated, then you must design in burst for the case of maximum surface shut-in on gaseous CO_2. The hydrostatic head caused by the fluid in the annulus must also be considered plus a safety factor of 1.125 which is the accepted norm. Some designers also consider a burst design that will withstand a pressure surge of a given size. This approach assumes that well control procedures will prevent unloading of the hole. The calculation procedure for this type assumption is tedious, generally does not improve the design significantly and is therefore not normally used in disposal well design. The general result is strengthening of the lower casing section and this is more correctly handled by the collapse design. Net burst loading is the difference in internal and external pressures at a point in the casing string. It is usually assumed that the external pressure is equivalent to the hydrostatic head of the column of drilling mud at that point. In cases where a loss circulation zone exists and it is doubtful that the annulus will stay full, the designer can assume no hydrostatic backup, to an arbitrary depth, to compensate for the loss of hydrostatic head. Abnormal conditions are best handled by the safety factors since trying to cover every eventuality would only increase the cost needlessly. After the final design is complete, draw an X-diagram (Figure 4) of depth vs rating and determine that the weakest point is at least half way down the string. This will insure that if failure does occur, due to abnormal conditions, that this failure is not near the surface.

FIGURE 4

ELLIPSE OF BIAXIAL YIELD STRESS

AFTER HOLMQUIST & NADAI



In collapse, the lower part of the string should be strong enough to support the hydrostatic pressure of the mud weight in which the pipe was cemented with no hydrostatic backup inside the casing plus a 1.125 safety factor below the cement top and 1.0 above the cement top if cement is not circulated to the surface.

Tensile design of this string will be significant because protective casing is typically the heaviest weight casing, has a large outside diameter (OD) and deeper setting depth. Tensile loads are reduced because of the buoyant force of the mud in which they are run. The cement top on the protective string is generally high and, thus, stability load changes do not increase tensile load appreciably. If the unsupported section is large or large differential pressures are expected after cementing the string, a check of stability load changes should be made to determine if setting conditions need to be altered or a pipe with greater tensile strength installed.

The clearance between the OD of the casing and the walls of the drilled hole is normally dictated by the casing size and the cement program, however, the requirements change with each well. For instance, in disposal well design, it is common to run 9 5/8" OD casing inside a 12 1/4" hole, however, 10 3/4" OD casing could be run inside the same hole if:

1. Mud conditions were excellent and it was lightweight,
2. The casing length was short,
3. The formations were competent,
4. The formations could withstand the frictional pressure drop, and
5. High cement success was not essential.

Since cement success is normally essential for disposal wells, the larger casing should only be run when special conditions exists. For disposal well systems, it is generally recommended that the hole size be a minimum of 1.5 inches larger than the casing diameter if the connections are to be welded or flush joint threads are used. If couplings are used, the hole size would be 1.5 inches larger than the coupling OD.

Primary Cementing

Probably the single most important factor in a well completion operation is to obtain a satisfactory primary cement job. The key to success is the proper placement of the cement completely around the casing (Allen and Robert, 1978). If the cement is properly placed around the casing, it serves the following three generally accepted purposes:

1. It isolates permeable zones at the wellbore to prevent or restrict fluid movement between formations;
2. It secures the pipe in the hole and supports the weight of the casing and;
3. It protects the casing against external corrosion by fluids contained in zones that the casing passes through.

To accomplish these objectives, it is necessary to place the most uniformly consistent (both in fluid properties and geometry) cement slurry possible between the casing and the formation and to enhance conditions affecting the eventual bond between the cement and the formation and the cement and the pipe (Figure 5). How to accomplish and to improve upon this seemingly simple task has been the topic of countless research papers, inventions, techniques, laboratory and field experiments throughout the industry. Millions of dollars have been spent in attempts to determine which factors may have a greater influence and how to manipulate which factors that have the greatest influence. Additional studies addressed the manipulation of the operational variables which affect those factors. There are two factors, related to borehole geomoetry, that are frequently not ranked high enough in the order of importance. These are thickness of the cement sheath and the effects of vertical deviation.

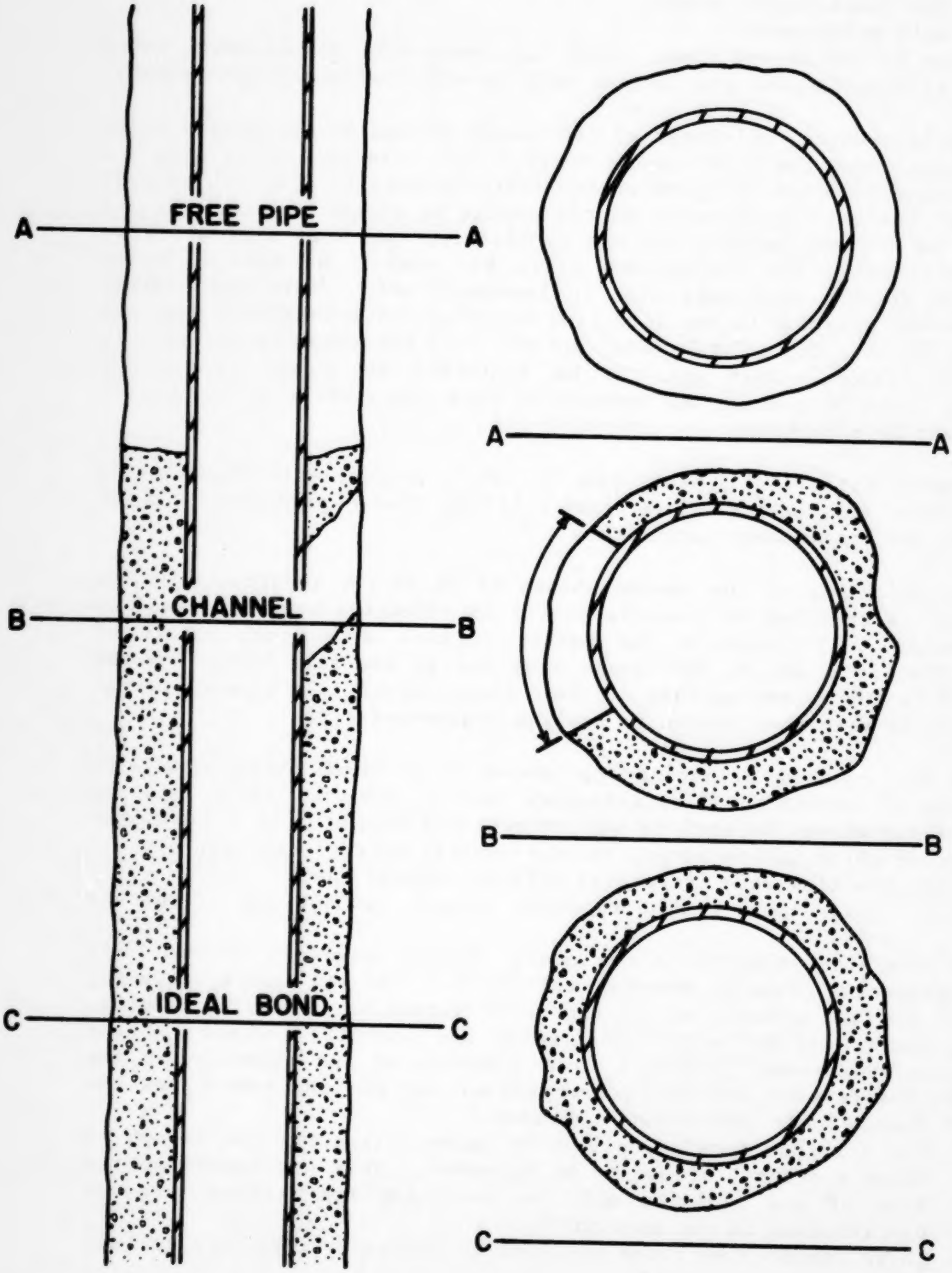
It is apparent that if a primary cement job is going to effectively isolate permeable zones and protect the outer steel casing surface from potentially corrosive formation fluids, it must provide a good hydraulic seal both in the horizontal and vertical directions. This means that the solidified cement mass must be free from contiguous channels or vugs and should form an impermeable shield around the casing that effectively blocks fluid from contacting the casing. It must also prevent migration up and down the casing/cement interface or along the formation/cement interface. In practice, a "perfect cement job" is impossible to achieve (Haut and Cook, 1979). In a laboratory, under ideal conditions, it is possible to obtain a "very good" cement job which in practicality satisfies all the major objectives. It must be recognized that under field conditions, even with the most stringent specifications, attention to detail, and sparing no expense, a "perfect" cement job cannot be achieved. Incomplete distribution of cement both in the annulus and vertically along the casing is usually referred to as "channeling" (Craft and Holden, 1962).

Haut & Crook (1979) recognized six basic factors which influence effective cement emplacement, namely:

- condition of the drilling fluid,
- pipe movement,
- pipe centralization,
- flow rate,
- volume of fluids flowed past a particular interval (contact time), and
- difference in density between the cement and drilling mud.

Borehole geometry affects primarily the annular flow rate and pipe centralization. As we shall see later in our discussion, the thickness of the cement sheath is dependent on several factors, some of which can be controlled and others which cannot. Considering practical benefits, effectiveness, technical feasibility and economics, the prudent operator designs his well to accomplish the very best primary cement job he can obtain. He then executes the procedure utilizing all those operational techniques known to enhance effective placement which he is practically able to do. All of these things are designed to increase displacement efficiency in one way or another.

FIGURE 5
TYPICAL CEMENT CONDITIONS


A
FREE PIPE
A
A
A
B
CHANNEL
B
B
B
C
IDEAL BOND
C
C
C

Smith (1976) lists factors that contribute to channeling types of cement failures as:

- Contact of pipe with formation,
- Poor mud properties,
- Failure to move pipe,
- Low displacement rates,
- Hole enlargement.

Early work by Howard and Clark (1948) has shown that displacement rates and centralization play the largest role in efficient mud displacement.

The theoretical thickness of the cement sheath at any single point is dependent upon the relationship between hole size and casing size and casing standoff which produces eccentricity (Figure 6). We can control very well the outside diameter of the casing we choose to run. We can control to a great extent, but not completely, the hole size which we make by choosing the appropriate drill bit size. We have a lesser degree of control over hole size in intervals which "wash out" larger than the bit size due to the formation reacting and interacting with the drilling fluid. These "wash outs" are the hole enlargements referred to by Smith (1976). Wash out can be minimized by proper control of drilling fluid properties and controlled flow rates while drilling, but it can not be eliminated.

Suman & Ellis (1977) state that "...it is particularly important to select hole and casing sizes and drilling fluid properties, which maximize mud displacement efficiency..."

The thickness of the cement sheath is of little importance if its integrity is breached by channels and if an effective hydraulic seal is not obtained. It would be far better to have a one-inch sheath of cement that is a solid, contiguous mass and is securely bonded to the pipe and to the formation than a 5-inch thick sheath full of mud pockets and channels with poor hydraulic sealing characteristics.

In the case of selecting the proper hole (drill bit) size with the goal of obtaining a satisfactory cement job, it is a case of optimization where "bigger" is not necessarily better. If a hole size is selected which is too large, two detrimental effects can occur:

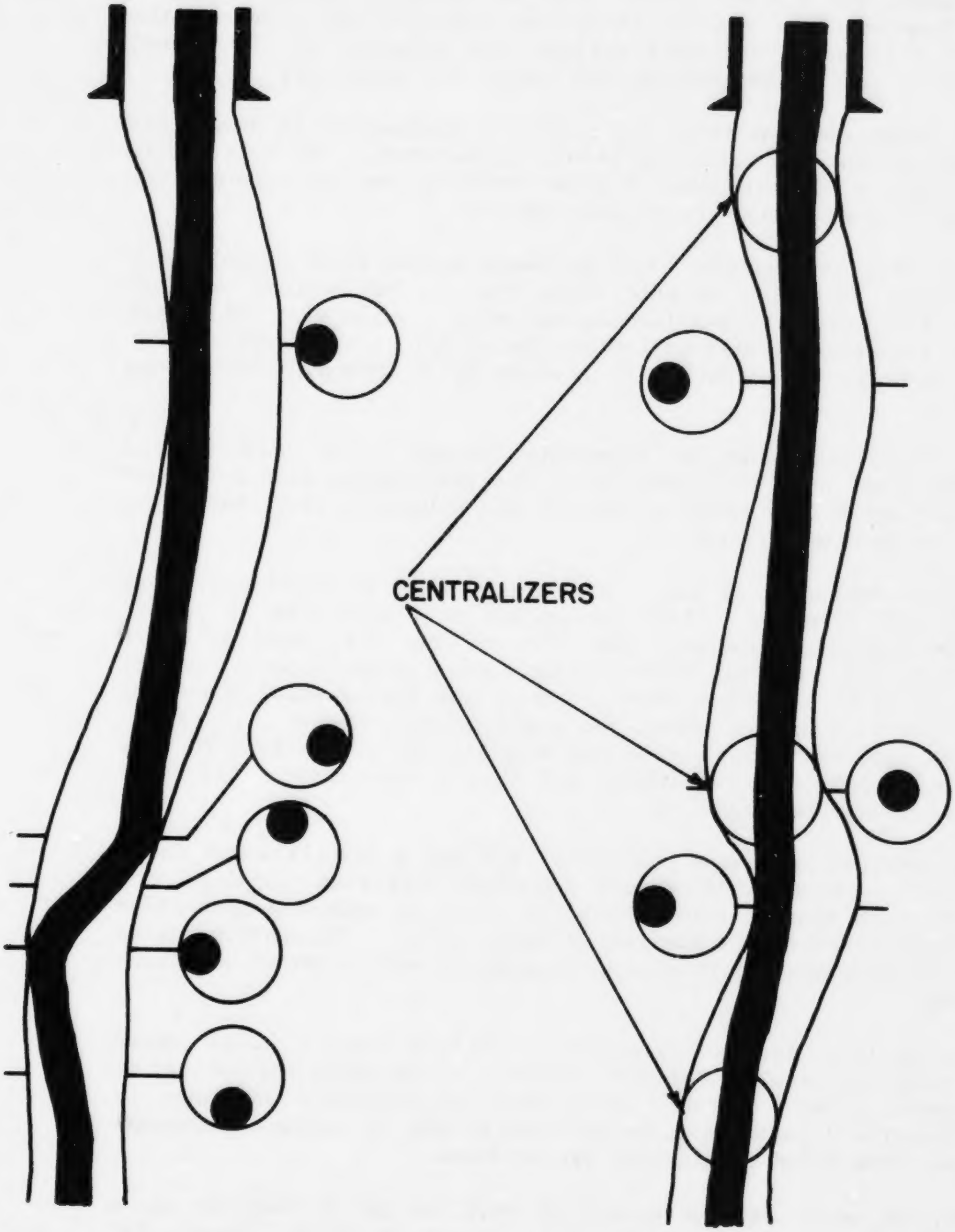
i) the flow rate (velocity) will be reduced, and
ii) centralizers and scratchers cannot contact the formation wall.

If the hole size selected is too small, direct formation contact will occur frequently, causing gross eccentricity. This may result in casing failure due to contact of casing with corrosive formation fluids. McLean, Manry, and Whitaker (1967) point out that the tendency of the cement to bypass mud (channel) is a function of the geometry of the annulus, the density and flow properties of the mud and cement and the rate of flow. Their experiment concluded:

- "in a narrow annulus, slightest decentralization was enough to allow a channel of mud to be bypassed. This was caused by the loss of mud fluidity and the resulting non-uniform pressure distribution in the annulus."
- "high cement flow rates appeared to favorably influence the mud displacement process."

FIGURE 6

CASING CONTACT


CENTRALIZERS

- mud displacement was maximized by fast cement pumping rates." Even when turbulent flow was not reached, better results were achieved when displacement was made at high flow rates.

It was once believed that circulating the cement slurry at a slow rate (plug flow) would enhance displacement. Haut and Crook's (1979) study and Brice and Holmes' (1963) work on turbulent flow techniques generally disprove this theory, hence the industry has accepted the theory that the faster and more uniform the velocity of the cement displacement of mud in the annulus, the better the cement job.

It is recognized that there are practical limitations to pump rates and therefore an optimum hole size should be selected. The hole should be small enough so that maximum annular velocity can be achieved but large enough to permit adequate centralization.

A great deal of practical field evidence exists which demonstrates that the proper selection of hole size, the use of centralizers and scratchers, pipe movement, preflushing and other displacement efficiency enhancement techniques really do improve the quality of a primary cement job. Two examples are detailed in studies by Hildebrandt (1971) and Holley (1976).

Other techniques such as cementing through drill pipe with a stab-in type float shoe (Cullihan, 1976) and preflushing with a viscous non-corrosive water base material (Ritter and McDaniel, 1969) have also been shown to enhance results.

From the standpoint of cost, it would be safe to predict that to drill a 12 1/4" hole for 9 5/8" casing and then underream it to 15" before cementing would increase the cost of the hole some 15 - 20 percent due to rig time and direct underreaming costs. Actual cement material cost would more than double. This cost differential would not be significant if a better cement job would result, however, it can be demonstrated that too large a hole can actually contribute to a poor or failed cement job and therefore, on this basis alone, the cost differential cannot be justified.

Harsh external corrosion conditions are not a justification for a large cement annulus. If severe external corrosion problems are anticipated, some consideration should be given to cathodic protection with impressed current as suggested by Watts (1968). Externally coated or lined tubular goods should also be considered when external corrosion is a problem.

There is very little presented in the literature which deals directly with the subject of the effects of borehole deviation on primary cement jobs. However, using what is available and what is inferred from the literature cited previously and the authors' personal experiences, some valid conclusions can be drawn.

First, we must ask the questions what is the definition of a "straight hole", and how straight is a "straight hole" (Figure 7)?

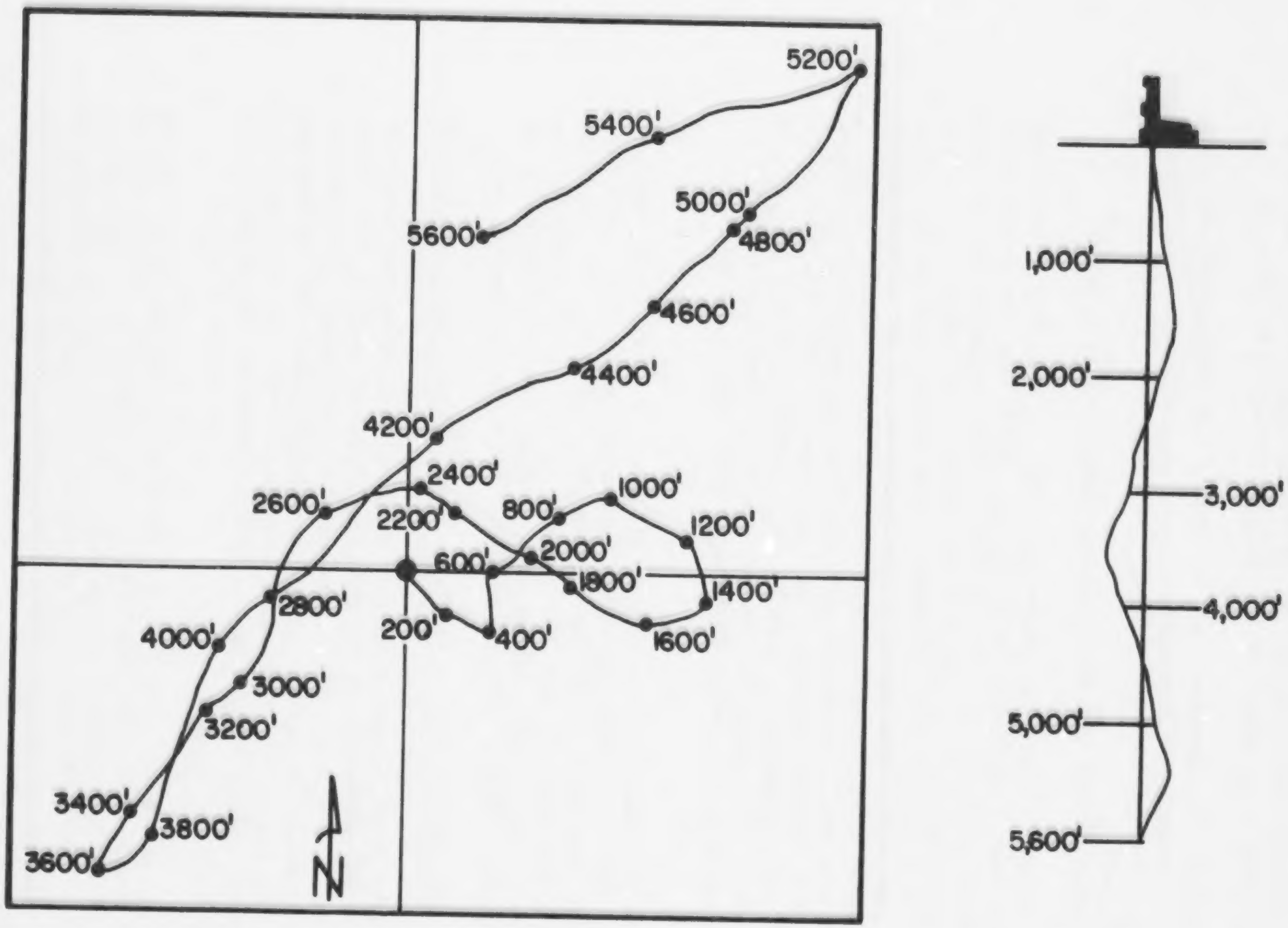

5200'
5400'
5000'
4800'
5600'
4600'
4400'
4200'
2400'
2600'
2200'
800'
1000'
1200'
2000'
600'
1800'
1400'
2800'
200'
400'
1600'
4000'
3000'
3200'
3400'
3800'
3600'
N
1,000'
2,000'
3,000'
4,000'
5,000'
5,600'

FIGURE 7
HOW STRAIGHT IS A STRAIGHT HOLE?

Historically, the term "straight hole" has been used to characterized a wellbore which is not intentionally deviated.

The International Association of Drilling Contractors in their publication "Drilling A Straight Hole" defines a straight hole as one that:

- Stays within the boundary of a "cone," as designated by the specifications of the operator,
- Does not change direction rapidly (no more than 3° per 100 feet of hole), and
- Has no large offset resulting in the formation of a ledge.

Myers and Sutko (1968) in their work on calculating forces on centralizers say that a straight hole is one where the deviation from the vertical never exceeds 3 1/2 degrees. This appears to be the petroleum industry accepted standard. The water well industry is more precise in their standard for plumbness and their definition of a straight hole. The U.S. Department of the Interior in its Ground Water Manual (1977) states that "....the usual standard for plumbness requires that the axis of the well casing not deviate from the vertical in excess of two-thirds the inside diameter of the casing per 100 feet of depth....". If this standard is applied to a 5000' disposal well to be completed with a 9 5/8 casing and we assume a 9" inside diameter, we would find the maximum allowable deviation from vertical would be 0.3 degrees.

The 0.3 degree maximum deviation allowed in the above example disagrees with the petroleum industry standard of 3 1/2" degrees, however, we must recognize that the water well industry is generally concerned with plumbness in shallow source wells as it effects the ability to run long, rigid submersible pumps and the effects on mechanical shafting. This condition does not exist in a disposal well.

Semantics and definitions aside, the true factors associated with borehole deviation are those which affect the efficient displacement of the drilling fluid with cement slurry. We have seen from our previous discussions that eccentricity of the casing/formation annulus (or casing standoff from the centerline) can contribute to channeling. Exaggerated standoff which results in actual direct contact between the casing and the formation can indeed be detrimental because there is no chance of getting any cement sheath between potentially corrosive formation fluids and the casing outer wall.

In reality, it is not the overall average deviation which should concern us but the severity of "doglegs" in the wellbore. A "dogleg" is defined in the drilling industry as an abrupt change in the angle and/or direction of the wellbore over a short distance. Combined changes in angle and direction tend to make the dogleg even more severe. Directional drilling companies have formulas from which a "dogleg severity index" can be calculated and experience has shown that if a certain value of dogleg severity is reached, hole problems can be expected. It is logical to expect that if doglegs occur in the wellbore over short intervals, centralizers will not be able to overcome the lateral forces imposed on the casing and that eccentricity or, actual wall contact can result. From the work done by Meyers and Sutko (1968), however, we see that if the deviation is smooth, a good centralizer program can reduce eccentricity to a minimum, in a 3 1/2 degree hole. Factors effecting angle or direction changes during drilling are:

- Weight applied to the bit
- Formation hardness
- Formation dip angle
- Bottom hole assembly configuration
- Normal bit walk
- Rotary speed.

Should the angle begin to show an increase, changes can be made in one or several of these variables in an attempt to stop the angle building or reduce the angle. These measures can be time consuming and thereby costly. Normally, little difficulty is encountered in holding maximum deviation to 3 degrees. Experience has shown that considerable difficulty can be expected if the tolerance allowed is only 1 degree. It is reasonable to expect that progress would be retarded and corrective measures may be necessary in every deep well attempted under this criteria. Retarded progress and corrective measures could increase the cost of the hole anywhere from 10 percent to 100 percent depending on how severe the problem is. Again however, increased cost is not the major factor. If industry was convinced that a better primary cement job could be obtained by restricting the deviation to a maximum of 1 degree from the vertical, this would be routine procedure because literally millions of dollars are spent on secondary or remedial cementing each year. However, experience has not generally shown this to be the case.

Average deviation from the vertical of 3 degrees to 3 1/2 degrees is an acceptable industry standard and in reality has very little effect on primary cementation provided the angle transitions are smooth and adequate provison for pipe centralization is made.

Inspection Techniques

Generally the methods used to inspect the integrity of a well's construction are referred to as Cased Hole Logs. They are run on an electric line and can provide significant information on the integrity of a well's construction. The most commoly used techniques will fall into one or more of the following generic catagories, viz:

- Radioactivity,
- Acoustic, and
- Specialized.

There is only one radioactive logging technique used to evaluate well construction. It is generally referred to as a Radioactive Tracer Survey. The Survey uses Iodine 131 as the radioactive source and a gamma ray tool to trace its movement. The well construction evaluation logs using acoustic signals are referred to as a Sonic Televiewer, Noise Log and Cement Bond Log. Schlumberger's new Cement Evaluation Tool also measures acoustic signals. The specialized logs commonly used to evaluate well construction are the Temperature Log, Directional Survey, Caliper Log, Flow Meter (Spinner Survey), Casing Collar Locator and and Casing Inspection Log. Generally, the name of the logging techniques indicate the parameter the log measures, e.g., the Temperature Log measures temperature changes, the Noise Log measures changes in sound, etc.

The Cement Bond Log uses an acoustic signal to evaluate the cement bonding to both the casing and the formation. Uncemented casing which is free to vibrate will transmit an acoustic signal. The pipe amplitude curve will have a high value, showing amplitude reduction at the couplings due to an increase in mass or wall thickness. Cemented casing, with cement well bonded to both casing and formation, is not free to vibrate, thus there will be very little acoustic signal at the characteristic arrival time for the pipe.

The evaluation of Cement Bond quality comprises three principle tasks:

i) Evaluation of the quality of cement to pipe bond;
ii) Evaluation of the quality of cement to formation bond; and
iii) Evaluation of the vertical extent of good bond quality.

The basic bond log tool consists of an acoustic transmitter (or sound source) and an acoustic receiver. The transmitter generates a sound pulse which travels by several different sound paths to the receiver. The three most obvious paths observed are the casing path, the formation path, and the fluid path (Figure 8). Because sound travels at different velocities in different mediums, the sound waves from the three paths normally arrive at three different times. The most common order for these arrivals are:

i) The casing arrival
ii) The formation arrival, and
iii) The fluid arrival

Of course the order of these arrivals can vary depending upon factors such as formation velocities, open hole size, casing size, and tool centralization.

When evaluating cement bond quality we normally evaluate the quality of approximately the first one inch of cement surrounding the casing by observing the characteristics of the casing arrivals. We evaluate the quality of the cement bond to the formation by analyzing the formation arrivals.

The evaluation of the quality of the cement bond to the casing is accomplished by analyzing the characteristics of the sound traveling through the casing. Sound travels from the transmitter, through the mud and strikes the casing, causing the casing to oscillate or ring. The ringing travels along the casing and back through the mud to the receiver. The magnitude of the sound received back at the receiver is dependent upon several factors:

- The magnitude of the original sound pulse
- The internal diameter of the casing
- The type fluid in the well
- The thickness of the casing wall

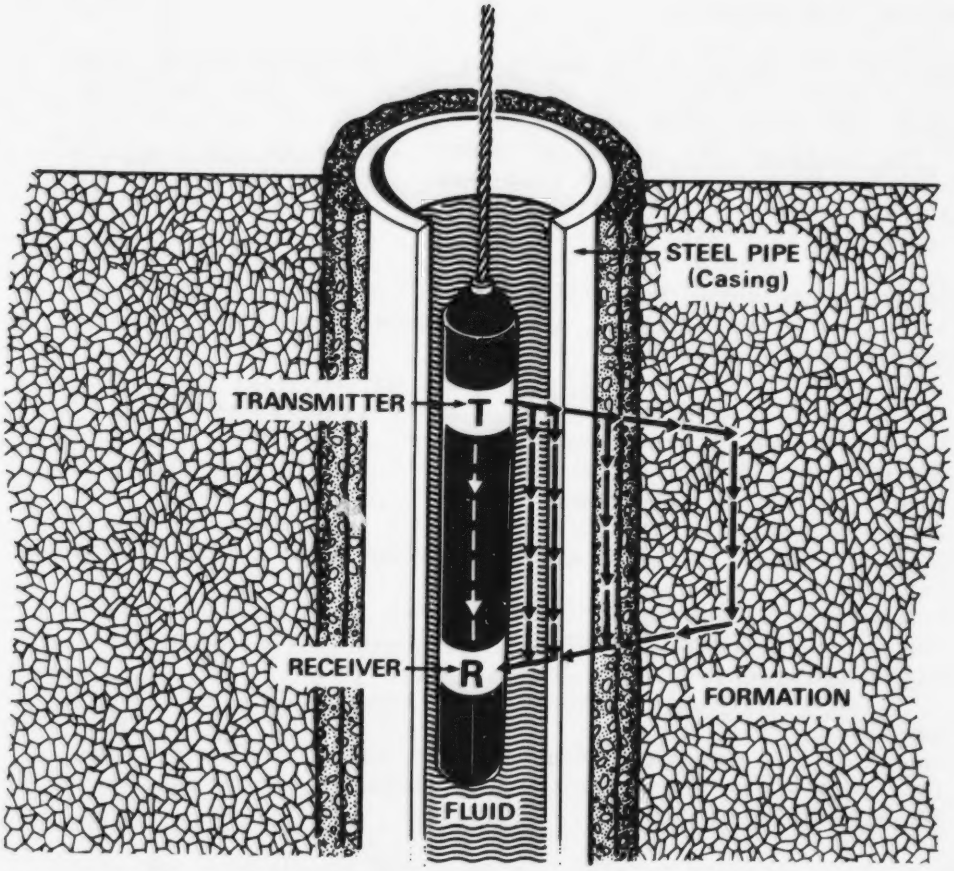

STEEL PIPE
(Casing)
TRANSMITTER
T
RECEIVER
R
FORMATION
FLUID

FIGURE 8
BASIC ACOUSTIC BOND LOG THEORY

- The amount of cement bonded to the casing
- The compressive strength of the cement bonded to the casing.

The effect of the sound pulse magnitude is adjusted to be a constant by calibrating the tools to a known standard. The effects of casing ID, type wellbore fluid and casing thickness on the magnitude of the received signal can be predicted based on empirical relationships derived from past experience.

The remaining two factors, the amount of cement bonded to the pipe and the compressive strength of that cement, are the only two non-predictable variables which affect the magnitude of the received signal. As you increase the amount of cement bonded to the pipe or increase the compressive strength of that cement, the magnitude of the received signal decreases because both properties impede the ability of the pipe to ring.

The important conclusion from this is that as the quality of the cement bonded to the pipe increases, the magnitude or amplitude of the received casing arrival decreases.

While realizing this important conclusion it is also important to recognize its limitations. These limitations are:

i) 75% of the received signal is a result of the first inch of cement surrounding the pipe;
ii) Cement sheath less than 3/4 of an inch thick does not fully attenuate the received signal, giving pessimistic cement evaluation;
iii) Cement sheath less than 3/4 of an inch thick bonded to both casing and formation acts as an infinitely thick sheath for high compressive strength formations;
iv) A microannulus gives pessimistic cement bond to casing evaluation; and
v) Hard rock formations (less than 60 microseconds/foot) interfere with casing arrivals invalidating amplitude based interpretation techniques.

Therefore it is imperative that you know the conditions for your particular well. Otherwise, as McGee and Vacca (1980) point out, "logging data may not always be representative of the actual cement-to-casing bond. In fact, improper logging and/or cementing techniques have frequently resulted in misleading cement bond interpretations resulting in unnecessary squeezing or completions having annular communication".

Froelich, Pittman and Seeman (1981) describe a new generation of cement and casing evaluation tool that they refer to as the Cement Evaluation Tool. This tool uses a different measurement principle. The tool is designed to investigate the cement radially and gives eight separate indications of cement placement and quality. In addition, casing diameter, casing roundness and tool eccentering are measured. Unlike the conventional cement bond log, this tool uses 8 transducers spiralled along its length with annular spacing of 45 degrees. With conventional cement bond logging tools, a sonic pulse leaves a transmitter, travels down the pipe, and is detected by a receiver spaced some distance away. With the Cement Evaluation tool, an ultrasonic pulse leaves the transducer, travels to the pipe and is reflected back to the same transducer. In this way, 8 well-defined segments of casing/cement are monitored at one time. A ninth transducer continually measures accoustic travel time in the casing mud column so that the other 8 transducer travel times can be converted to distance measurements. The use of multiple transducers provides the tool with some unique capabilities that are not possible with a conventional cement bond logging tool. These included a measurement of tool eccentricity, casing condition and casing deviation. However, the most significant ability of the tool is its ability to locate cement channels behind the casing.

All wells have a geothermal gradient that is caused by the continuous flow of heat from the interior of the earth. The magnitude of the gradient depends upon the difference in temperature between the surface and the heat source in the interior of the earth and upon the thermal conductivities of the materials in between. A Temperature Log is a measure of this geothermal gradient plus other manmade effects that cause deviation from normal conditions.

The temperature log is probably the most commonly run specialized log. The temperature log is used to:

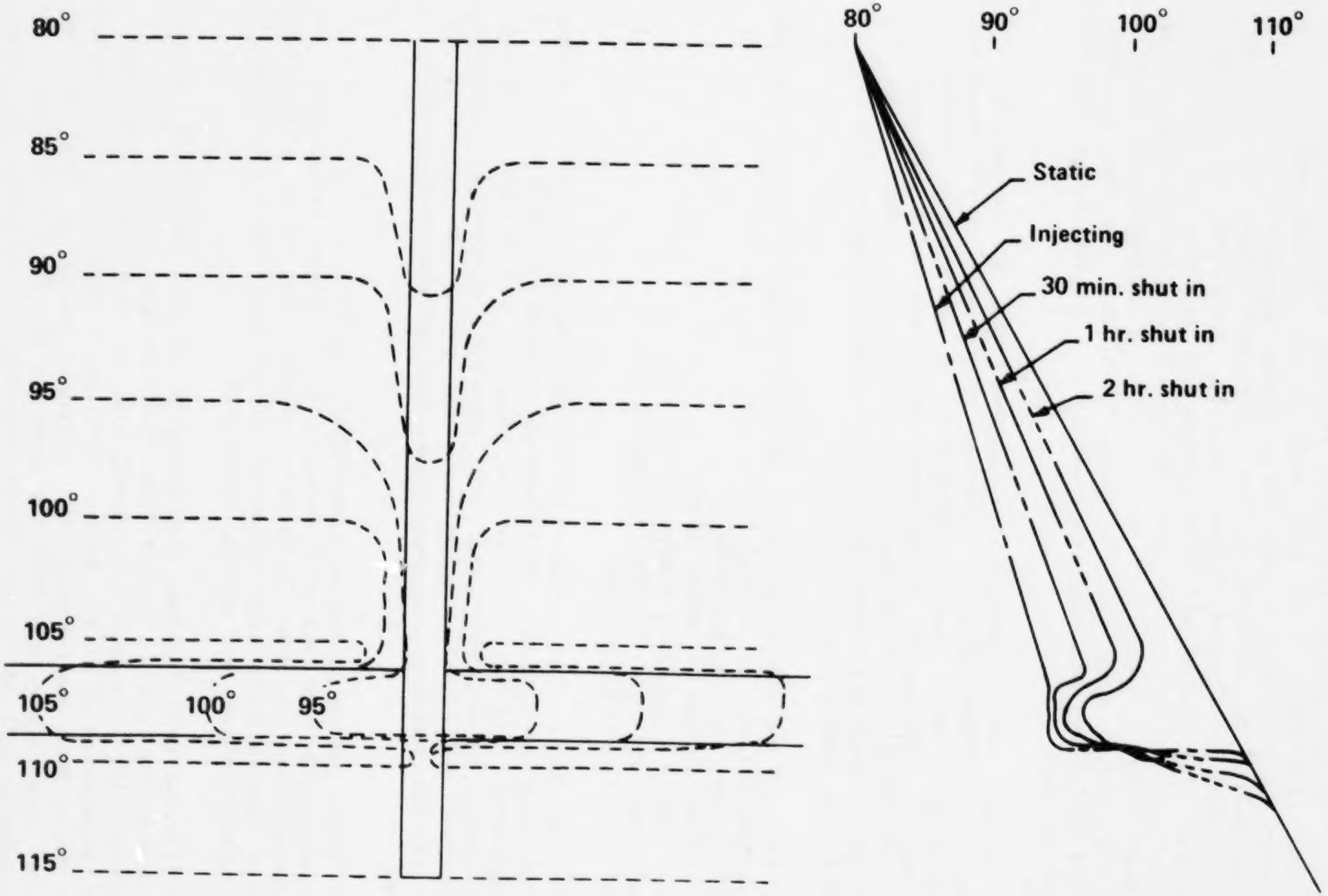
i) locate the top of cement;
ii) find tubing or casing leaks;
iii) locate channeling behind the casing;
iv) evaluate loss circulation zones; and
v) run injection surveys.

It is of utmost importance that the well be stabilized to normal conditions prior to logging. This does not necessarily mean shut-in. A base temperature log is run with the well in its normal condition. After changing the wellbore conditions (shut-in, producing, injecting, etc.) several additional logging runs are made. By comparing the changes of the various log anomalies, the problem can usually be diagnosed.

The basic understanding of temperature log interpretation is that fluid channelling from below is usually at a higher temperature and fluid channelling from above is usually cooler. This is not always true in disposal well systems since there are wells injecting either high and low temperature fluids and in some cases both.

In disposal well systems, the injection of cold fluids into a well will distort the normal temperature distribution (Figure 9). For illustration, let us assume a disposal well is receiving fluids with a surface temperature of 80°F. As the fluid moves downhole, it is warmed by the formations surrounding the borehole until it reaches some stabilized temperature, say 95°F for purposes of this illustration. No further warming will occur, and while injection is occuring, the entire injection zone will take 95°F fluid. The injected cooler fluid will displace the warmer in situ formation waters thus creating a thermal gradient within the injection zone.

FIGURE 9
SCHEMATIC DIAGRAM AND LOGS ON A COLD FLUID INJECTION WELL


80°
85°
90°
95°
100°
105°
105°
100°
95°
110°
115°
80°
90°
100°
110°
Static
Injecting
30 min. shut in
1 hr. shut in
2 hr. shut in

When injection is stopped, heat begins to flow back to the wellbore. The return to normal temperature will be relatively quick where the cold fluid has not penetrated any formation. The injection zone itself, however, will take a longer time to return to normal temperature because of the warm fluid displacement. The length of time required to return to normal will depend upon the length of the injection period. An important point on the shut-in logs, that may be run, is the "boundary effect". Just above the permeable zone near the wellbore, heat is flowing down to the permeable zone and horizontally to the wellbore. Therefore, this area would warm up more slowly than that further up the hole giving rise to the increased appearance of "smearing" at the boundaries. This boundary effect is what gives the differential temperature logs their upthrown appearance and erroneous interpretations of channeling above perforations. The differential temperature log would show its greatest deflection to the left in this boundary zone, and all it indicates is that the fluid went somewhere below.

The effect of injection of hot fluids would be essentially the same as that discussed for cold fluids except opposite in direction. The wellbore would be heated only to a shallow lateral distance where the hot fluid has not penetrated a permeable formation. The section where hot fluid flowed down the wellbore would cool soon after injection is stopped, with the injection zone taking considerably longer to return to normal temperature.

The recommended general approach for running temperature logs, on any type of well, is to run a log with the well in its stabilized normal condition producing or injecting. Then run a series of logs after the well condition is changed. This would mean a stabilized flowing log and a one, two, and four-hour shut-in logs in a producing well. The same procedure would be followed in an injection well; run an injecting log and a series of shut-in logs.

Another type specialty log is the noise log which monitors various portions of the audible frequency range. The various frequencies of sound can be related to specific fluid movement. The frequencies monitored are selected so that the lower frequencies associated with the noise from the rig, logging truck, and cable do not interfere with the survey. Very high frequencies are not used because of their great reduction in amplitude.

The Noise Log can be used to locate fluid movement in channels behind pipe, which, in some cases, cannot be detected by temperature logs. The noise log can also find leaks in the casing and detect sand movement in channels of entry into the well bore. One or two phase flow is delineated by the various frequencies monitored. It is also possible to estimate the amount of fluid entry into the casing.

The disadvantage of the noise log is that it must be recorded with the tool stationary for about 3 minutes at each level and time in some cases could become excessive.

The Spinner Survey is another specialized log and provides a means of measuring the down-hole flow rate of a well (Figure 10). The spinner is primarily used to gauge the production or injection from each set of perforations. The velocity of the fluid flow causes a propeller to rotate, which in turn generates a small current. The magnitude or frequency of this current, recorded at the surface, is related to down-hole flow rate.

Spinner surveys are extremely accurate in one phase flow. However multiphase flow can lead to erroneous interpretation because of the differences in the rotational energy of each phase.

The casing of a well can be surveyed for corrosion by the use of electronic calipers, which detect changes in the wall thickness by electro-magnetic changes. The corrosion detectors are similar to the collar locator but much more sensitive. The casing inspection tools have an exciter coil through which an alternating current flows and a receiver coil which picks up the resulting current flow through the casing. The phase difference between the transmitted and received current is related to the wall thickness of the pipe. A different coil geometry, spacing, and winding, are necessary for each different casing size and weight; therefore, false anomalies can occur if there is a change in the casing grade.

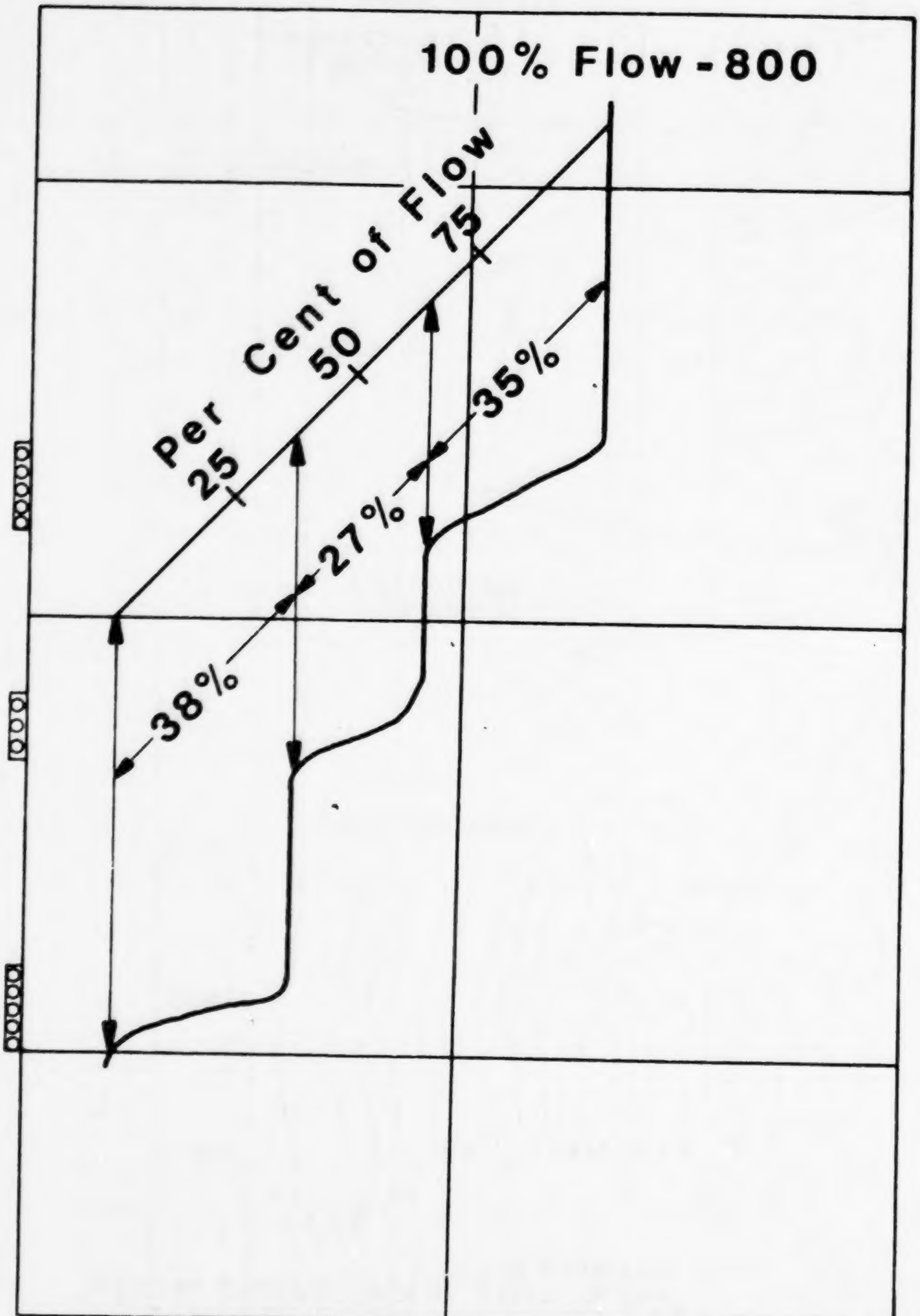
Internal corrosion of the casing can occasionally be detected by running a multi-arm caliper. This tool has several arms and each arm continuously monitors changes in the casing ID as the tool is pulled up the hole. These changes are recorded at the surface and can usually be related to the condition of the casings inter wall.

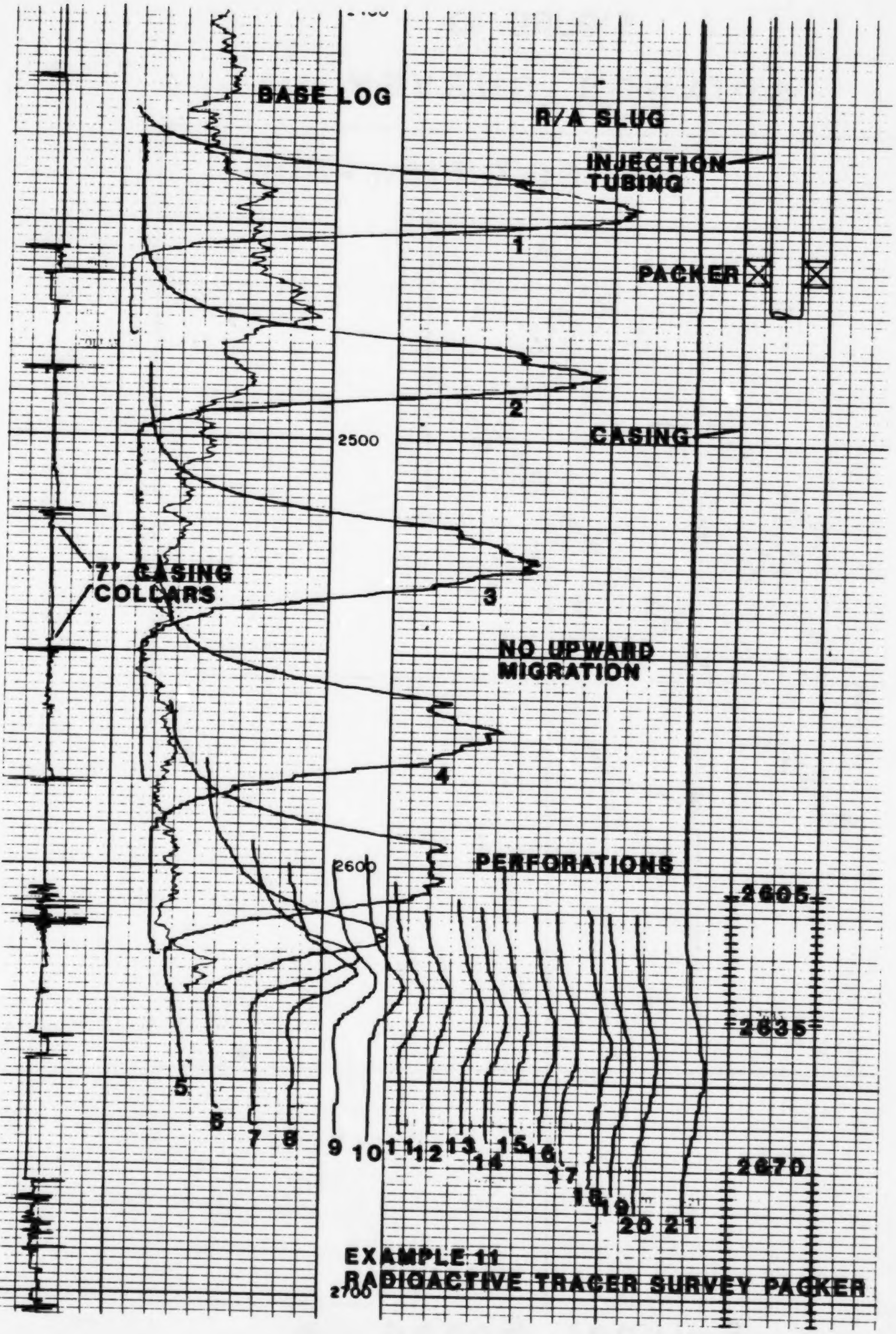
The aforementioned logs are generally approved by both the EPA and State UIC programs to evaluate a well's mechanical integrity. However, radioactive tracer surveys require special permission. Like the temperature log, a radioactive tracer survey is highly interpretive and subject to the method used. However if proper techniquies are used, this log is one of the best tools available to identify fluid movement from the injection zone along the outside of the casing.
To determine if vertical migration is occurring from the injection zone along the outside of the casing, the following procedure is generally used (Figure 11).

A base or reference gamma ray log is run to determine the natural background radiation. For the purposes of this illustration let us assume that we want to determine if any fluid movement is present above the perforations in a disposal well. The procedure will be to run the base log and then, while injecting into the well at a slow rate, release a small amount of Iodine 131 several feet above the injection interval. To trace the movement of the radioactive material, make successive logging runs across the perforations screen or open hole. These successive runs will show the movement of the tracer fluid into the zone and if no vertical migration is occurring, the tracer fluid will dissipate into the disposal zone. However, if migration were occurring along the outside of the casing, the gamma ray tool would detect the Iodine 131 outside the casing above the receiving zone.

FIGURE 10

SPINNER SURVEY





EXAMPLE 11
RADIOACTIVE TRACER SURVEY PACKER

Summary

The technology is available to properly design and construct disposal well systems that will not contaminate our ground water supply. Proper isolation of the zones penetrated is of primary importance and can be accomplished through the use of properly designed casing programs, primary cement jobs and inspection techniques.

The casing must have sufficient strength to control the loading conditions that will be encountered. This includes not only the body of the pipe itself, it also includes the connections. Like a chain, a casing design is only as good as its weakest link. The degree of uncertainty (safety factor) is always a problem. However, each company or individual will usually have their own design factors that are based on experience. The designer should first develop the well plan and then determine the degree of uncertainty he has. From these data he can then determine the design factors to be used. Without exception a casing design should always address collapse, burst, tension and compression loading.

A good casing design is almost worthless if it is not properly cemented in the drilled hole. Therefore, attaining the best possible primary cement job on casing is of paramount importance. The cost involved to do it right the first time is far less than attempting to repair it after the fact. Whenever possible the designer should:

i) Design the hole and casing sizes to achieve maximum displacement efficiency;
ii) Keep the hole straight to within 3 degrees maximum deviation; but most importantly, keep the hole smooth with minimum dogleg severity;
iii) Use optimally spaced centralizers and scratchers on the casing; string.
iv) Move the casing while cementing (rotate and/or reciprocate).
v) Use a preflush ahead of the cement slurry;
vi) On large diameter casings, cement through an inner-string using a stab-in type float shoe;
vii) Reduce mud weight, plastic viscosity, and yield point to the minimum allowable limit and circulate prior to placement of the cement;
viii) Displace the cement at maximum attainable rates; and
ix) Run caliper logs and calculate as accurately as possible cement volumes and use excess cement to increase contact time.

The more of these individual items that can be accomplished, the better the chances will be of obtaining a satisfactory primary cement job.

Many wireline methods and tools are available for the detection of a multitude of wellbore problems. The following is a general tabulation of the logs or tools that can be used to locate the problem identified.

To determine the fluid flow profile across the perforations or completion interval.

- Radioactive Tracer Survey

- Spinner Survey

To detect fluid movement behind the casing, run a
- Temperature Log
- Noise Log
- Radioactive Tracer Survey

For locating and detecting casing, tubing and packer leaks, run
- Radioactive Tracer Survey
- Temperature Log
- Noise Log
- Spinner Survey

For monitoring casing corrosion, run one of the
- Various types of corrosion detector logs
- Casing Caliper logs

In conclusion, disposal wells provide an important method for the removal of wastes from man's biosphere and it can be done without contamination of the ground water system. Proper design, proper construction techniques and post-construction monitoring are the key factors.

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Evaluation of Confining Layers for Containment of Injected Wastewater

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ABSTRACT

The general technical requirements of a suitable wastewater injection well site include:

1. An injection interval that is sufficiently thick, extensive, porous and permeable to accept wastewater at the necessary rate at safe injection pressures.
2. Overlying and underlying strata (confining beds) sufficiently thick, extensive and impermeable to confine waste to the injection interval.
3. The absence of faults or extensive joints or unplugged or improperly plugged abandoned wells that would permit escape of injected wastewater from the injection interval into adjacent aquifers.

Since the advent of the use of injection wells for wastewater disposal, one of the more difficult technical issues has been the determination of the adequacy of confinement of injected liquids within the injection reservoir. This may be a particular problem when the confining interval is relatively thin, shallow, of questionable permeability, is affected by local

structural or stratigraphic geologic features, or is penetrated by abandoned wells that may be improperly plugged. 2

Assessment of the adequacy of a confining interval usually involves qualitative evaluation of the stratigraphy, structural geology and hydrogeology of the site but some quantitative or semiquantitative methods are also available for testing and modelling the behavior of confining strata, including the vertical extent of hydraulic fracture growth and the rate of flow of injected wastewater through them.

INTRODUCTION

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Since the advent of the use of injection wells for wastewater disposal, one of the more difficult technical issues has been the determination of the adequacy of confinement of injected liquids within the injection reservoir. In many cases, this is not an issue, because the injection interval is so deep and the confining interval so thick and impermeable that there is no need for detailed analysis. However, in other instances, the confining interval is:

1. Relatively thin,
2. Shallow,
3. Of questionable permeability,
4. Affected by local structural or stratigraphic geologic features,
5. Penetrated by abandoned wells that may be improperly plugged,
6. Affected by some combination of 1-5.

When permits have been requested for injection wells at sites where confining layer effectiveness has been uncertain for one or more of the reasons given above, it has, sometimes, proven difficult to make the regulatory decision. This is probably because of the lack of quantitative criteria upon which to base the decision. That is, values for confining layer thickness, permeability, etc. have not been generally established.

This paper summarizes the geologic and engineering factors involved in and procedures available for the evaluation of confining strata for containment of injected wastewater. The theory of hydraulic fracturing is briefly reviewed and the possible effect of hydraulic fracturing on confining layer integrity discussed. Procedures for calculation of the flow of wastewater through confining layers are given.

GEOLOGIC CRITERIA

Knowledge of the geologic and hydrologic characteristics of the subsurface environment at an injection well site and in the surrounding region is fundamental to the evaluation of the suitability of the site for wastewater injection and to the design, construction, operation and monitoring of injection wells. In defining the geologic environment, the subsurface rock units that are present are described in terms of their lithology, thickness, areal distribution, structural configuration and engineering properties.

Injection and Confining Intervals

Vertical sequences of rocks that occur in the subsurface are conventionally subdivided by geologists into groups, formations, and members, in descending hierarchy. That is, members are subdivisions of formations and formations subdivisions of groups. Use of these terms implies mappable (traceable) rock subdivisions, based on mineralogy, fossil content, or other recognizable characteristics. However, such subdivisions may or may not be entirely suitable when discussing subsurface flow systems, because the engineering properties of porosity and permeability often do not respect geologic boundaries. This problem was long ago recognized by ground water hydrologists who developed the terms aquifer, aquiclude, aquitard, and aquifuge to describe rock subdivisions in terms of their capacity to hold and transmit water. An aquifer is defined as a formation, group of formations, or part of a formation that contains sufficient saturated permeable material to yield significant quantities of water to wells and springs. Conversely, an aquiclude stores water, but does not transmit significant amounts. An aquitard lies between the two previously defined types in that it transmits enough water to be regionally significant, but not enough to supply a well. An aquifuge neither contains nor transmits water (Davis and DeWiest, 1966). These same terms could be applied in discussing wastewater injection, and sometimes are; but other terms are more commonly used. Actual or potential receiving aquifers are commonly called the injection intervals; zones, units, or reservoirs and the intervening strata are referred to as confining intervals (aquicludes) or semi-confining intervals (aquitards). The basement sequence of igneous or metamorphic rock that lies beneath the sedimentary rock cover is generally non-porous and impermeable (aquifuge).

Rock Types

Rocks are described in terms of their origin and their lithology, the latter characteristic being defined by their composition and texture. While nearly all rock types can, under favorable circumstances, be capable of acting as injection intervals, sedimentary rocks, particularly those deposited in a marine environment, are most likely to have suitable geologic and engineering characteristics. These characteristics are sufficient porosity, permeability, thickness, and areal extent to permit the rock to act as a liquid-storage reservoir at safe injection pressures.

Sandstone, limestone, and dolomite are types of sedimentary rock commonly porous and permeable enough in the unfractured state to be suitable injection reservoirs. Naturally fractured limestone, dolomite, shale, and other rocks may also be satisfactory.

Unfractured shale, clay, siltstone, anhydrite, gypsum, and salt have been found to provide good seals against upward or downward flow of fluids. Limestone and dolomite may also be satisfactory confining beds; but these rocks commonly contain fractures or solution channels, and their adequacy must be determined in each case.

Stratigraphy

Stratigraphy or stratigraphic geology is the study of the composition, sequence, thickness, age and correlation of the rocks in a region. A stratigraphic study of injection and confining intervals is a fundamental step in the evaluation of a site for wastewater injection. Such studies utilize all available relevant surface and subsurface geologic data which are displayed in the form of stratigraphic columns, cross sections, fence diagrams, thickness (isopach) maps and facies maps. Qualitative judgements concerning confining intervals can be made from stratigraphic studies and such studies provide the geologic framework for the development of quantitative models that will be discussed later.

Structural Geology

Structural geology is concerned with the folding and fracturing of rocks and the geographic distribution of these features.

Structural geologic characteristics of a region and, on a smaller scale, of a particular site are significant because of their role in influencing subsurface fluid flow and the engineering properties of rocks. As with stratigraphy, large-scale structural geologic features are depicted by the use of cross sections and maps.

In the case of confining intervals, fractures, which include faults and joints, are of particular concern. Faults are fractures in the rock sequence along which there has been displacement of the two sides relative to one another. Such fractures may range from inches to miles in length, and displacements are of comparable magnitudes. Faults may occur singly or in systems so complex that it is not possible to completely define them.

It is known, through experience, that faults may act either as barriers to fluid movement or as channels for fluid movement. However, little is known in detail about how and why some faults are barriers and others are flow channels. In theory, no fault in a sedimentary rock sequence will be an absolute barrier, but a fault may be of so much lower permeability than the aquifer it cuts that it is, for practical purposes, a barrier. Since it will seldom be possible for a geologist to initially state whether a fault is a barrier or a flow path, it would be appropriate to consider any significant fault to be a flow path for purposes of preliminary evaluation of its importance. If, as a consequence of this initial assumption, the fault would be an environmental hazard it would then be necessary to either abandon the project or to test the fault directly by pumping or injection testing. A significant fault might be defined as one that is of sufficient length, displacement, and vertical persistence to provide a means of travel for injected wastewater to an undesirable location. This question is one that will have to be answered by qualified geologists and engineers on the basis of their best judgement, after a review of the data for a particular site.

Fractures also exist along which there has been no movement. This type of fracture may be referred to as a crack or joint to distinguish it from a fault. Cracks and joints are important sources of porosity and permeability in some aquifers but can be undesirable when they channel fluids rapidly away from an injection well in a single direction or where they provide flow paths through confining strata. The presence and nature of fractures is determined by examination of rock cores obtained during drilling, by well logging and testing methods and from experience with other deep wells drilled in the same region.

Earth Stresses and Faulting

Vertical stress in a rock at any depth is simply the pressure exerted by the weight of fluid-saturated rock overlying that depth.

Horizontal stresses cannot be calculated without performing hydraulic fracturing and determining the orientation of the induced fractures but can be estimated from an analysis of existing faults. The fault type is an indication of the stresses that existed when the fault movement occurred and does not necessarily indicate the present state of stress. However, in regions where faulting is very recent, the faulting pattern may indicate present stresses, for example the Gulf Coast area.

Normal faulting indicates that the least principal stress, when the faulting occurred, was horizontal and that the maximum principal stress was vertical and equal to the pressure exerted by the overburden.

Thrust or reverse faulting indicates that the least principal stress was vertical and equal to the overburden pressure. The maximum principal stress was horizontal and greater than the overburden pressure.

In a sedimentary rock sequence, the total normal vertical stress increases with depth of burial, under increasing thicknesses of rock and fluid. It is commonly assumed, and the validity of the assumption can easily be verified, that the normal vertical stress increases at an average of about 1 psi/ft of depth. The lateral stresses may be greater or less than the vertical stress, depending on geologic conditions. In areas where crustal rocks are being actively compressed, lateral stresses may exceed vertical ones. In areas where crustal rocks are not in active compression, lateral stresses should be less than the vertical stress. The basis of estimating lateral stress prior to drilling of a well is hydraulic fracturing data from nearby wells and/or knowledge of the tectonic state of the region in which the well is located. The tectonic state of various regions is only now being determined. For example, Kehle (1964) concluded, as a result of hydraulic fracturing data from four wells, that the stresses at the well locations in Oklahoma and Texas were representative of an area that was tectonically in a relaxed state. In contrast, Sbar and Sykes (1973) characterized much of the eastern and north-central United States as being in a state of active tectonic compression.

In order to predict the pressure at which hydraulic fracturing or fault movement would be expected to occur, it is necessary to estimate the state of stress at the depth of the injection horizon. On the other hand, determination of the actual fracturing pressure allows computation of the state of stress (Kehle, 1964).

The general equation for total normal stress across an arbitrary plane in a porous medium is (Hubbert and Willis, 1972):

$$S = p + \sigma \qquad [F/L^2] \qquad (1)$$

where S = total stress

p = fluid pressure

σ = effective or intergranular normal stress.

Effective stress, as defined by Equation 1 is the stress available to resist hydraulic fracturing or the stress across a fault plane that acts to prevent movement on that fault. The equation shows that, if total stress remains constant, an increase in fluid pressure reduces the effective stress and a decrease in fluid pressure increases effective stress. When the effective stress is reduced to zero by fluid injection, hydraulic fracturing occurs. In the presence of a fault, along which shear stress already exists, fault movement will occur before normal stresses across the fault plane are reduced to zero.

Further discussion concerning the state of stress and hydraulic fracturing will be presented in the section on hydraulic fracturing.

Engineering Properties

In order to make a quantitative evaluation of the mechanical response of the subsurface environment to wastewater injection, the engineering properties of the rocks must be determined or estimated. Properties classed here as engineering include porosity, permeability, and the elastic parameters, such as compressibility and Poisson's ratio.

Porosity

Porosity is defined as:

$$\phi = \frac{V_v}{V_t} \text{ (dimensionless)} \qquad (2)$$

where ϕ = porosity, expressed as a decimal fraction

V_v = volume of voids

V_t = total volume of rock sample.

Porosity is also commonly expressed as a percentage. Porosity may be a total porosity or effective porosity. Total porosity is a measure of all void space; effective porosity is based on the volume of interconnected voids. Effective porosity better defines the hydraulic properties of a rock unit, since only interconnected porosity is available to fluids flowing through the rock. The difference between total and effective porosity is often not known, or it may be small, but the distinction should be kept in mind.

Porosity may also be classified as primary or secondary. Primary porosity includes original intergranular or intercrystalline pores and the porosity associated with fossils, bedding planes, and so forth. Secondary porosity results from fractures, solution channels, and from recrystallization and dolomitization.

The amount of porosity in confining strata is not as important as the form of porosity. Shales have a relatively large total intergranular porosity, but the pores are small and flow through them difficult. Fractures, on the other hand, may contribute only a small amount of porosity but the fractures, if open, may be relatively transmissive flow paths.

Permeability

The permeability of a rock is a measure of its capacity to transmit a fluid under an applied potential gradient. As with porosity, intergranular permeability is influenced by the grain properties of rocks that are composed of grains (sands, sandstones, siltstones, shales, etc.). However, whereas porosity is not theoretically dependent on grain size, permeability is strongly dependent on this property. The smaller the grains, the larger will be the surface area exposed to the flowing fluid. Since it is the frictional resistance of the surface area that lowers the flow rate, the smaller the grain size the lower the permeability. Shales, which are formed from extremely small grains, have almost no permeability. This is why shales are selected as confining intervals. As with effective porosity, permeability also results from interconnected solution channels and fractures as well as from interconnected intergranular spaces.

Permeability values from core samples of units used for wastewater injection or petroleum production range from several darcys to less than one millidarcy, but an average value of less than 10 millidarcys for an overall interval would be considered to be very low, whereas a value of 100-1,000 millidarcys would be good to very good. Shales, which are considered to be suitable confining strata, have permeabilities in the order of 10^{-3} to 10^{-6} millidarcys, or thousands of times less than an adequate injection interval.

Compressibility

The compressibility of an elastic medium is defined as:

$$\beta = \frac{-\partial V}{V \partial p} \qquad [F/L^2]^{-1} \qquad (3)$$

where β = compressibility of medium [pressure^{-1}]

V = volume

p = pressure

The compressibility of an aquifer includes the compressibility of the aquifer skeleton and that of the contained fluids. To account for the compressibility of both the fluid and the aquifer, petroleum engineers often arbitrarily use a compressibility (c), which ranges from 5×10^{-6} to 10×10^{-6} psi^{-1} as compared with the compressibility of water alone which is about 3×10^{-6} psi^{-1}. Compressibility enters into flow equations where unsteady or transient flow is concerned.

Poisson's Ratio

Competent rocks are generally considered to behave as elastic materials over the range of conditions involved here. When stressed either in tension or compression, the rock's dimensions change and these dimensional changes are referred to as strain. The ratio of lateral to axial strain or deformation of an elastic material is Poisson's ratio. This ratio may be as high as 0.45 for highly elastic rocks and between 0.20 and 0.30 for brittle dense rocks such as limestones and dolomites. Poisson's ratio appears in equations for hydraulic fracturing pressure and may be an important determinant in the vertical height of fracture growth, as will be discussed later.

CONFINING LAYER EVALUATION PROCEDURES

The geologic and engineering criteria relating to the suitability of a rock unit for confinement of inejcted wastewater have been discussed. In this section, the methods available for confining layer evaluation during the drilling and testing of injection wells are outlined.

Logs

Individuals have different concepts of what is meant by a borehole or well log, because of the variety of types of logs that are used. It is probably best to consider a log as any tabular record or graphical portrayal of drilling conditions or subsurface features observed in a borehole.

One possible classification of the types of logs would be:

1. Sample (cuttings and core) logs
2. Driller's logs
3. Drilling time logs
4. Geophysical logs
   - A. Electrical logs
   - B. Elastic wave logs
   - C. Radiation logs
   - D. Other
5. Miscellaneous logs

Well logs provide data on the stratigraphic sequence at a site and some geophysical logs yield useful engineering properties, particularly porosity, resistivity and Poisson's ratio. Many of the logs yield information concerning the existence of natural fractures. Some of the log categories are discussed below in more detail.

Driller's Logs

In the early days of drilling, the driller's log was the principal well record kept. It recorded the types of formations encountered, any pertinent fluid flows or oil and gas shows observed, and other related operational remarks. The current rotary driller's log is filled out daily by each driller as a record of the operations, materials used, and progress which occurred during his working hours (tour). Driller's logs can be very important in determining the source of problems that occur during well operation and in resolving uncertainties with regard to exactly how subsurface facilities are installed. In the evalution of a confining interval, reported incidents of lost circulation would be particularly important and would indicate the presence of highly permeable and porous zones resulting from solution activity or fracturing.

Drilling Time Logs

A drilling time log is often kept by the driller when hole depth approaches a zone of particular interest. Such a record is quite useful for precisely locating formations or porous zones. Abrupt changes in drilling rate will immediately indicate a change in lithology although the cuttings may not reach the surface for some time. Drilling rate is importantly controlled by lithology and rock physical properties. In carbonate rocks, in particular, a sudden increase in drilling rate may mean the presence of a porous and permeable zone that would be undesirable in a confining interval.

Geophysical Logs

After a well has been drilled, a variety of down-hole logging tools are available that can be used to produce a record of the geophysical properties of the formations penetrated and their contained fluids. The geophysical properties that are measured include electrical resistivity and conductivity, ability to transmit and reflect sonic energy, natural radioactivity, hydrogen ion content, temperature, density, etc. These geophysical properties are then interpreted in terms of lithology, porosity, fluid content, and chemistry. Such logs are extremely useful in confining layer evaluation. For example, the gamma-ray is a semiquantitative indicator of shale percentage and the acoustic or sonic logs detect porosity, yield rock elastic properties and locate fractured and vugular zones. Because the variety of methods and applications is so extensive, the reader is referred to the literature and to the oil-field logging service companies for further information.

Miscellaneous Logs

Among the logs classified here as miscellaneous are caliper logs, which measure borehole diameter; dipmeter logs, which measure the angle of dip of beds penetrated by the well and borehole televiewer logs, which provide an image of the borehole wall as produced by the reflection of sound waves emited from a downhole source. These and perhaps other techniques may be useful in confining layer evaluation. As with geophysical logs, the reader is referred to the literature and to the companies that supply the logging services for further information.

Injection or Pump Testing

Field pump testing (or injection testing) methods have been developed by groundwater hydrologists and have been refined for use in the analysis of gas storage reservoirs that are well suited for the evaluation of confining layer adequacy in wastewater injection applications. This application was suggested by Witherspoon and Neuman (1972); but, as far as is known, has not been attempted. While the methodology is potentially very effective for confining layer evaluation, it requires the drilling of at least one observation well to a depth nearly as great as that of the injection well itself. That requirement is economically very limiting and probably explains why the technology has not yet been used in conjunction with wastewater injection.

Because the mathematics are somewhat involved, the details of the Witherspoon-Neuman (1972) confining-layer evaluation procedure will not be repeated here. An example provided by them should be sufficient to show the potential of the method. The example involves evaluation of a 150 foot thick sandstone aquifer at a depth of about 1,200 feet overlain by a 400 foot thick confining layer of limestone and dolomite. Core analyses gave an average porosity of 14 percent and permeability of 140 millidarcys for the sandstone and a porosity of about 2 percent and a permeability of 10^{-4} to 10 millidarcys for the bottom 50 feet of the confining layer. However, cores showed vertical or near-vertical fractures at several intervals in the carbonate section. A pumping test was performed in a well completed

through the sandstone using an average rate of 40 gpm for three days. Fluid 11
levels were measured in an observation well 750 feet from the pumping well completed in the sandstone and in an observation well 962 feet from the pumping well completed 50 feet above the sandstone in the carbonate confining interval. The results are shown in Figure 1. The water level decline measured in the confining layer (aquitard well) shows it to be much more permeable than the core analyses would indicate. Quantitative evaluation of the data yielded a permeability value of 0.6 millidarcys for the lower carbonate interval. This could only be the result of the effects of fracturing. Further testing with observation wells placed at higher levels in the carbonate interval revealed that the entire interval was fractured and not suited as a confining stratum for gas storage. It would obviously not be suited for wastewater confinement. An additional analysis of the significance of the quantitative value of confining layer permeability, as obtained from such testing, is given in the section on "Flow Through Confining Layers."

A simpler method of evaluating confining layers by pump or injection testing is to install observation wells in the first aquifer above the confining interval and to monitor for water level or pressure changes during injection into or pumping from the injection unit. While simpler, this method is not as sensitive as one in which the observation wells are installed within the confining interval.

Water Composition Analysis

Samples of water from subsurface formations can be obtained from deep wells, before they are completed, by use of formation testing devices, by swabbing, and by gaslift. At times, formation pressures will bring water to the surface from artesian aquifers.

Water samples will not normally be obtainable from confining layers, except perhaps by extraction from cores. However, water samples from aquifers above and below a confining interval can provide evidence of hydraulic separation (or connection) of the aquifers. Table 1 gives water analyses for three aquifers at the Garfield gas storage area, Illinois. The Eau Claire confining interval is between the Mt. Simon and Galesville aquifers and an extensive carbonate sequence is between the Galesville and St. Peter aquifers. Because the water qualities in the three aquifers are so dissimilar, it could be inferred that they are hydraulically separated by the intervening confining beds.

HYDRAULIC FRACTURING AND CONFINING LAYER INTEGRITY

Hydraulic fracturing of reservoir formations is widely applied as a means of increasing near-wellbore porosity and permeability and is an acceptable stimulation method, when carried out in a controlled manner. Massive hydraulic fracturing or long-term injection at pressures above the fracture pressure are not, however, acceptable, in wastewater injection well construction and operation since either might result in propagation of a vertical fracture through the confining interval. The theory of

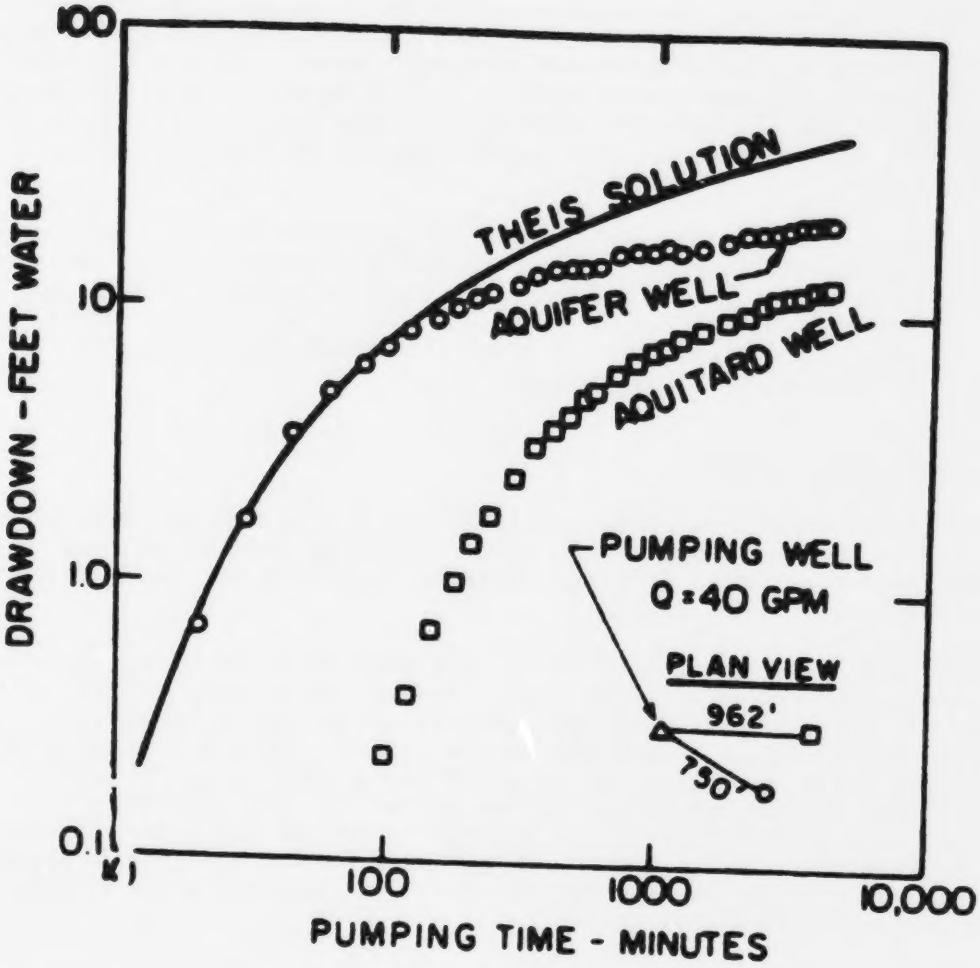

Figure 1. Results of pumping test with leaky aquifer (Witherspoon and Neuman, 1972).

Table 1 Analyses for water samples obtained from the Mt. Simon, Galesville and St. Peter aquifers at the Garfield gas storage area, Illinois (Katz and Coats, 1968). Values in parts per million.

Well	Fordyce 1	Scheurer 1	Musser 1	Fordyce 3	Clark 1
Aquifer	Mt. Simon	Mt. Simon	Mt. Simon	Galesville	St. Peter
pH	7.5	7.5	7.9	7.3	7.8
Carbon dioxide	15.	17.	6.	19.	274
Bicarbonate	240	282	250	238	274
Chlorides	10,200	6,995	11,400	1,800	473
Hardness	2,350	1,750	3,040	968	234
Calcium	676	504	880	254	45
Magnesium	150	120	200	80	30
Sulfates	660	500	590	460	145
Iron	14	17	47	4.5	2.5
Total Dissolved Solids	17,901	13,174	18,416	3,785	1,243

hydraulic fracturing and its implications with regard to confining layer integrity are discussed below.

Hydraulic fracturing is the process in which fluid pressure is applied to the exposed reservoir rock until failure or fracturing occurs. After failure of the reservoir rock, a sustained application of fluid pressure extends the crevice or fracture outward from the wellbore. The fluid used to transmit the hydraulic pressure to the reservoir rock is called the fracturing fluid. Since fractures tend to heal or close when the pressure is allowed to return to normal, propping agents such as sand or glass beads are injected with the fluid to keep the fractures open during deliberate well stimulation operations. The theory of hydraulic fracturing is summarized by Howard and Fast (1970). A fracture is initiated at the wellbore wall when the induced hydraulic pressure exceeds the combined resistances of formation tensile strength and rock compressive stress. It is often assumed, because of historic observations of fracturing projects, that rock has preexisting fracture systems and very little or no tensile strength, so that the only stress to be overcome is the least principal rock stress.

In estimating the fluid pressure at which hydraulic fracturing will occur, one of two conditions is usually assumed:

1. That the least principal stress is less than the vertical lithostatic stress caused by the rock column. In this case fractures are assumed to be vertical.

2. That the vertical lithostatic stress is the least principal stress. In this case fractures will be horizontal.

In the first case, the minimum bottom-hole pressure required to initiate a hydraulic fracture can be estimated from (Hubbert & Willis, 1972):

$$p_f \cong \frac{S + 2p_o}{3} \quad (F/L^2) \qquad (4)$$

where

p_f = fracture initiation pressure

S = total lithostatic stress

p_o = formation fluid pressure

The hydraulic fracturing gradient, that is, the injection pressure required per foot of depth to initiate hydraulic fractures, can be estimated by entering representative unit values into Equation 4. The unit values for S and p_o are, respectively, 1.0 and 0.45 psi/ft. This yields a p_f gradient of 0.63 psi/ft as a minimum value for initiation of hydraulic fractures. This situation implies a minimum lateral earth stress. As the lateral stresses increase, the bottom-hole fracture initiation pressure also increases up to a limiting value of 1.0 psi/ft. Actually, fracture pressures may exceed 1.0 psi/ft when the rocks have significant tensile strength and no inherent fractures that pass through the well bore.

In any particular case, injection tests can be run on the well to determine what the actual fracture pressure is. In the absence of any specific data, arbitrary limitations of from 0.5 to 1.0 psi per foot of depth have been imposed on operating injection wells. Regional experience should be used as a criterion in establishing an aribtrary limit, since regional tectonic conditions and fluid pressure gradients dictate what a safe limit will be.

Equation 4 is a simplified one in which an average value for Poisson's ratio is included in the constants. A more useful equation, for confining layer evaluation is (Eaton, 1969):

$$\frac{p_f}{D} = \left(\frac{S}{D} - \frac{p_o}{D}\right)\left(\frac{\nu}{1-\nu}\right) + \frac{p_o}{D} \qquad (5)$$

where

D = depth to interval being evaluated

ν = Poisson's ratio

It is essential that fractures be contained largely to the injection zone since break-out of the fracture into overlying or underlying formations could have serious consequences. From Equation 5 it is apparent that:

1. In an open hole where the fracture pressure gradient (p_f/D) remains constant with depth, hydraulic fractures should propogate upward, since the pressure to initiate or extend a fracture becomes smaller as depth becomes less.

2. As Poisson's ratio increases, the FPG increases. For example, given a typical p_o/D value of 0.45 psi/ft and a typical S/D value of 1.0 psi/ft, then the relation between FPG and ν is:

when ν =	0.10	.20	.30	.40
FPG =	0.51	.59	.69	.82 psi/ft

The relation between Poisson's ratio and the FPG is considered to be a basis for the limitation of the vertical growth of hydraulic fractures. Hunt et al (1981) state that "The hydraulic fracture once initiated will extend in a plane normal to the minimum stress (vertically) until the crack reaches a boundary zone; a boundary zone could be a region of increased stress ..." Poisson's ratio appears to be a suitable indicator of fracture boundaries in that a high value of ν produces high stress. According to Hunt et al (1981) shales are often considered as barriers to vertical fracture growth but all shales are not barriers. Those with high ν values were found by Hunt et al (1981) to act as barriers while those with lower ν values did not. Determination of the in-situ values of ν for prediction of fracture gradients and fracture barriers was accomplished by Hunt and others (1981) using the Schlumberger Well Services Digital Sonic log and SARABAND suite of logs.

Three recent case histories of the application of the fracture gradient method of predicting fracturing height are given by Labudovic´ (1984). Labudovic´ used borehole logs for calculation of Poisson's ratio which was entered into Equation 5 for determination of the fracture pressure gradient. This information in combination with resistivity and spontaneous potential logs was used to predict fracture height. Observed fracture heights compared reasonably well with predictions. 16

The studies discussed above indicate the possibility of identifying confining layers that would resist hydraulic fracturing from borehole geophysical logs. It appears, however, that this methodology is still in the developmental state and should be used with caution in conjunction with injection well projects.

FLOW THROUGH CONFINING LAYERS

The entire purpose of the evaluation of a confining layer or layers for containment of injected wastewater is to provide assurance against vertical migration of wastewater or saline water from the injection unit into overlying fresh water bearing aquifers. Such vertical movement could occur as a result of:

1. Intergranular flow through unbreached confining strata,
2. Flow through naturally fractured or faulted confining strata,
3. Flow through confining strata with solution porosity and, permeability
4. Flow through artificially fractured confining strata,
5. Flow through abandoned unplugged or improperly plugged wells.

Flow under each of these conditions is discussed below.

Natural Intergranular Flow

If the confining layer is a clastic sedimentary rock, that is, it is composed of discrete sedimentary particles and is unfractured, then fluid flow will be through intergranular spaces. Shales and siltstones and gradations between them are examples of such rocks. Flow through the intercrystalline spaces in chemically deposited rocks such as limestones is also intergranular flow.

Darcy's law for the flow of water through a granular rock is:

$$v = \frac{\overline{K}\ \Delta P}{\mu\ \Delta L} \qquad (6)$$

where:

v = darcy velocity [cm/sec]

$\bar{K}$ = permeability [darcys]

μ = viscosity [centipoise]

ΔP = pressure differential across the flow distance [atmosphere]

ΔL = flow distance [centimeters]

Furthermore, the actual average intergranular liquid velocity is:

$$\bar{v} = \frac{v}{\phi_e} \tag{7}$$

where:

$\bar{v}$ = intergranular velocity [cm/sec]

ϕ_e = effective porosity

The fluid pressure in a reservoir into which liquid is being injected is greatest at the injection well face and declines approximately logarithmically away from the borehole. The pressure will also vary with time, increasing as injection continues or declining if injection ceases or the rate is reduced. For purposes of illustration, however, a maximum constant pressure can be assumed to exist at the injection well in order to calculate the rate at which wastewater might be moving vertically through a confining layer. Table 2 shows illustrative rates of movement expressed in feet/year, for a range of pressures and permeabilities. For these cases, the confining layer was assumed to be 100 feet thick, the effective porosity to be 10 percent and the viscosity one centipoise.

A vertical wastewater travel of 114 feet/year would probably be unacceptably high but the assumed permeability in this case (1×10^{-3} darcys) is at the upper end of values that one might encounter in a confining stratum, whereas a value of 1×10^{-9} darcys is at the other extreme. The travel rates shown in Table 2 are only meant to be illustrative and not comprehensive. Additional computations can be made, quite simply, if so desired.

No mention has, so far, been made of the chemical quality of water that might be forced to flow from the injection unit to an overlying aquifer by injection pressure buildup. If the vertical flow is within the radius of spread of wastewater in the injection unit, then the water that would initially enter the overlying aquifer would be from the confining stratum and that would eventually be followed by injected wastewater. Outside of the radius of spread of the wastewater, the water initially entering the overlying aquifer would be from the confining stratum eventually to be followed by natural water from the injection unit.

An example of the type of analysis described above was documented by Hickey (1984) for an injection well facility at St. Petersburg, Florida.

Table 2 Distance of vertical travel of injected wastewater in feet/year through a 100-foot-thick confining stratum with various permeabilities and pressure gradients. The effective porosity was assumed to be 10 percent and the viscosity to be one centipoise.

	ΔP (psi)			
K (darcys)	500	1,000	2,000	5,000
1×10^{-3}	114 ft/yr	228 ft/yr	456 ft/yr	570 ft/yr
1×10^{-5}	1.14	2.28	4.56	5.7
1×10^{-7}	1.14×10^{-2}	2.28×10^{-2}	4.56×10^{-2}	5.7×10^{-2}
1×10^{-9}	1.14×10^{-4}	2.28×10^{-4}	4.56×10^{-4}	5.7×10^{-4}

The stratigraphic and hydrogeologic section for the St. Petersburg injection site is given in Figure 2. Injection was carried out during a period of 32 months beginning in September, 1979, to determine the impact of long-term injection. During the first year of the test, the mean injection rate was 2,750 gpm through a single well into injection zone C and by a postulated wellbore interconnection into injection zone B. Hickey (1984) calculated the vertical velocities in the semiconfining bed between zones A and B and also the pressures at various monitor well locations using analytical equations similar to those given here.

A much more powerful and revealing method of analysis, for a problem as complex as that at St. Petersburg, is the use of a numerical model such as the U.S. Geological Survey Saline Water Injection Program or SWIP (INTERCOMP Resource Development and Engineering, Inc., 1976). The SWIP model was used in this study to obtain predicted pressures, vertical velocities and saline water displacements in the injection zones and semiconfining units at St. Petersburg for comparison with Hickey's (1984) results.

Table 3 shows a comparison of pressures at selected monitoring points at the St. Petersburg injection well site as calculated by Hickey (1984) using analytical equations and as calculated for this study by use of the SWIP model. No field data were given by Hickey to allow comparison between observed and calculated pressures.

Figure 3 shows upward vertical velocities of pore water in the confining layer between zones B and A. The velocities are those computed by the SWIP model to exist after one year of injection of 2,750 gpm into zones B and C as explained above. For comparison, Hickey (1984) calculated velocities, using analytical equations, of 0.005 to 0.05 ft/day at a distance of about 733 ft. from the injection well and 0.01 to 0.1 ft/day immediately adjacent to the injection well. Figure 4 shows the calculated concentration fraction (C/C_o) of dissolved chemicals in zone A as forced upward from the semiconfining layer into zone A from injection into zone B. For example, if the seminconfining layer were to contain water with a chloride concentration of 13,000 mg/ℓ (Fig. 2), then the chloride concentration in the lower 20 feet of zone A would be increased by about:

$$(13{,}000\ \text{mg}/\ell)(0.056) = 728\ \text{mg}/\ell$$

immediately adjacent to the injection well after one year of injection.

Naturally Fractured Strata

The effect of naturally occurring fractures in a confining layer is, commonly, to greatly increase the permeability and to create a secondary form of porosity which is small in magnitude but effectively interconnected. The net result is that wastewater transport through a fractured caprock may be tens or hundreds of times more rapid than through the same rock in the unfractured state.

The analysis of flow through fractured rock is often treated no differently than flow through granular media. This approach is probably acceptable in many cases; but, where fractures are widely spaced, are of wide aperture

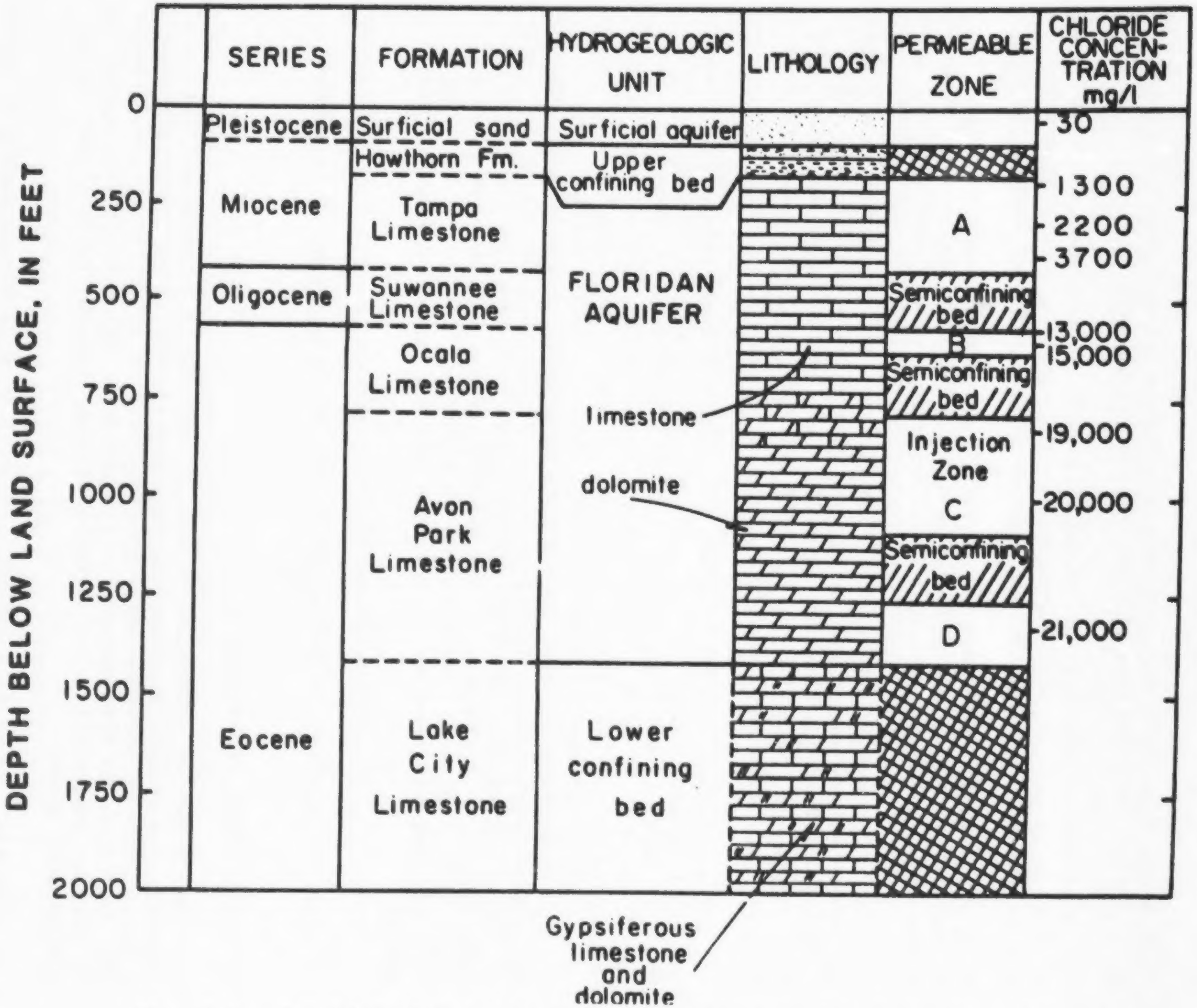
SERIES
FORMATION
HYDROGEOLOGIC UNIT
LITHOLOGY
PERMEABLE ZONE
CHLORIDE CONCENTRATION mg/l
DEPTH BELOW LAND SURFACE, IN FEET
0
250
500
750
1000
1250
1500
1750
2000
Pleistocene
Miocene
Oligocene
Eocene
Surficial sand
Hawthorn Fm.
Tampa Limestone
Suwannee Limestone
Ocala Limestone
Avon Park Limestone
Lake City Limestone
Surficial aquifer
Upper confining bed
FLORIDAN AQUIFER
limestone
dolomite
Lower confining bed
A
Semiconfining bed
B
Semiconfining bed
Injection Zone C
Semiconfining bed
D
30
1300
2200
3700
13,000
15,000
19,000
20,000
21,000
Gypsiferous limestone and dolomite

Figure 2. Stratigraphic and hydrogeologic section, St. Petersburg injection site (Hickey, 1984).

Table 3 Comparison of pressure buildups at observation well locations at St. Petersburg, Florida as calculated by Hickey (1984) and as calculated for this study using SWIP.

Observation Well	Distance from Injection Well	Pressure Buildup From Hickey	 From SWIP
A3	0	4	2.9
B6	45	1.6	2.5
B7	66	2.4	2
B8	100	0.5	1.0
B9	115	0.1	.08
B1	660	0.1	1.0
B2	660	0.1	1.0
B3	660	1	1.2
B4	660	1.4	1.3
B5	660	0.5	.3
C2	1300	0.8	.8
C3	1390	0.3	1.8

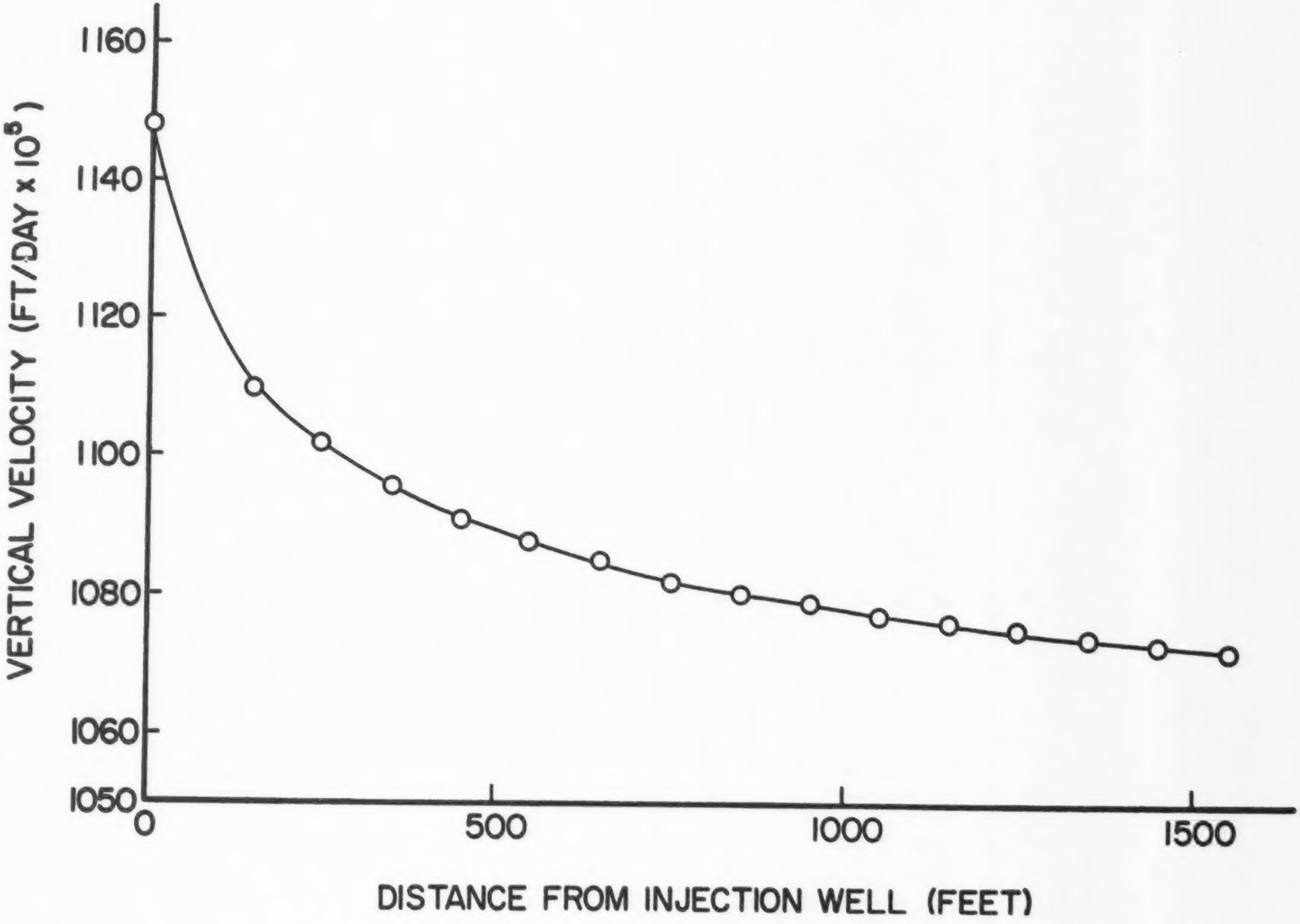

Figure 3. Vertical upward velocities of pore water in the confining layer between injection zones A and B at St. Petersburg, Florida, as calculated by computer model.

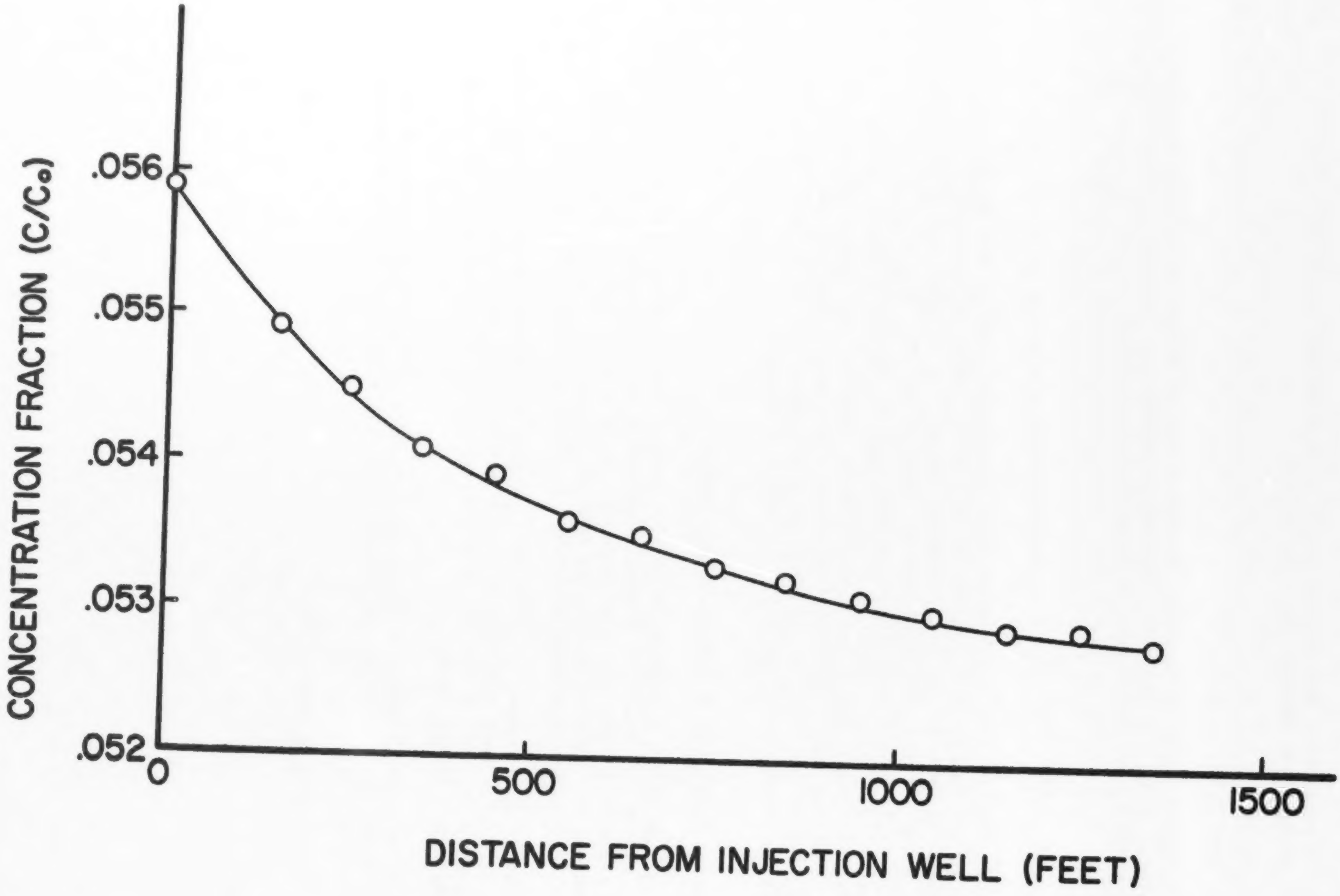

Figure 4. Concentration fraction C/C_0 of dissolved chemicals in zone A as forced upward from the semiconfining layer into zone A by injection into zone B at St. Petersburg, Florida.

or have a particular directional trend other mathematical methods may be needed (Freeze and Cherry, 1979, p. 73-74).

Strata With Solution Porosity

Solution porosity is often developed in conjunction with fractures and in soluble rocks and the above statements concerning fracture porosity will, thus, commonly be true where solution porosity exists.

Artificially Fractured Strata

Artificial fractures differ from naturally occurring ones in that they are more likely to be represented by a single vertical or horizontal fracture as opposed to a network of fractures. Also, if the artificial fracture has been deliberately induced, it will be propped open with sand, glass beads or other propping agents. Equations have been developed to predict fluid pressure distribution patterns in the vicinity of a well injection into single vertical or a horizontal fracture (Gringarten and others, 1974; Gringarten and Ramey, 1974). However, it is not likely that such equations would find application to the flow of fluid through a confining layer, since injection would be precluded if it were known that the confining layer was breached by an induced fracture.

Abandoned Unplugged or Poorly Plugged Wells

An abandoned unplugged or poorly plugged well, penetrating the injection unit, within the radius of pressure influence of the injection well can act as a point leak from the injection unit to overlying aquifers. This is, of course, the main basis for the area of review determination required in the UIC regulations promulgated under the Safe Drinking Water Act.

While development of analytic equations to predict rates of interaquifer flow through a well bore should not be particularly difficult, this has apparently not been done. A simplified approach can be taken to obtain an estimate of leakage rates.

First, the pressure in the injection unit at the location of the unplugged borehole would be calculated using the following equations (Warner, and others, 1979):

$$\Delta P = 162.6 \frac{q}{Kh} \log \frac{\bar{K}t}{70.4\phi\mu c r^2} \qquad (8)$$

where:

Parameter or Variable	Symbol	Practical Units
Compressibility	c	psi^{-1}
porosity	ϕ	decimal fraction
reservoir thickness	h	ft
permeability	$\bar{K}$	md
viscosity	μ	cp
pressure increase	ΔP	psi
flow rate	q	B/D
radial distance	r	ft
time	t	D

Since ΔP, the reservoir pressure increase will vary with time, a large time should be used, e.g. a year, after which relatively little change in ΔP will occur.

Now, Equation 8 can be rewritten as:

$$q = \frac{(\Delta P)\bar{K}h}{162.6\ \mu \log \left[\frac{\bar{K}t}{70.4\phi\mu c r^2}\right]} \qquad (9)$$

Then, if it is assumed that:

1. ΔP will remain constant (or change very little),
2. The unplugged well bore itself provides no resistance to flow,
3. The unplugged well bore is open through the length of the injection unit and the overlying aquifer,

a value of q can be calculated.

The procedure would be to calculate q for both the injection unit and the overlying aquifer, using the appropriate values for each unit, and to then select the smaller value of q as the estimate since the lowest value will be the controlling one. It is clear that q will decrease with time, as it should, if ΔP remains constant. It should be stressed that the method suggested here for calculating leakage through an unplugged wellbore is intended only as a means of obtaining an indication of the possible impact of such a point of leakage through a confining layer and is not presented as being precisely defendable.

CONCLUSIONS

The general geological characteristics of an acceptable confining interval are:

1. Sufficient thickness, lateral extent and impermeability to confine waste to the injection interval. 26

2. The absence of faults or extensive joints that would permit escape of injected wastewater from the injection interval into adjacent aquifers.

Additionally, confining intervals must not be breached by abandoned unplugged or improperly plugged wells or by induced hydraulic fractures that would permit the vertical escape of wastewater into adjacent aquifers.

An extensive array of techniques and tools are available for the geologic and engineering evaluation of confining strata. The extent to which the available methodology is applied, in any particular case, is a matter of the judgement of the well owner and of the regulatory authorities and will involve economics as well as science and engineering.

ACKNOWLEDGEMENTS

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BIOGRAPHICAL SKETCHES

Don L. Warner is Dean of the School of Mines and Metallurgy, and is a Professor of Geological Engineering at the University of Missouri-Rolla. He received his degree in Geological Engineering in 1956 and his Master of Science in 1961, both from the Colorado School of Mines at Golden. He obtained his Ph.D. in Engineering Science in 1964 from the University of California-Berkeley. He has authored numerous technical papers and reports and is consultant to many governmental organizations and private companies. His professional interests are in quantitative ground-water hydrology, ground-water quality protection, and water pollution and environmental studies.

Talib Syed is a petroleum engineer with Engineering Enterprises, Inc. in Norman, Oklahoma since July 1983. He previously worked as a petroleum engineer for the Arabian-American Oil Company in Saudi Arabia from 1974 to 1979 primarily in production completions and workovers, both onshore and offshore. Syed graduated with a degree in Chemical Engineering from the University of Madras, India in 1971 and earned a Masters of Petroleum Engineering from the University of Oklahoma, Norman, in 1983. He is presently primarily involved in studies related to the Underground Injection Control Program for the U.S. Environmental Protection Agency.

Richard Tinlin has over 22 years experience in the fields of hydrogeology and geophysics. He has managed several major groundwater related contracts for the U.S. Environmental Protection Agency and currently is managing a contract funded by the EPA which provides technical assistance to the EPA for the implementation of the Underground Injection Control Program. He has served as an editor for Groundwater Monitoring Review and has a number of publications on problems rleated to groundwater contamination.

TECHNOLOGICAL CONSIDERATIONS IN CLASS I INJECTION WELLS

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Abstract

Technological improvements have advanced the "state of art" construction techniques for Class I waste disposal wells. Items such as well design, installation, completion and monitoring have been standardized under the Federal Underground Injection Control (UIC) program to provide greater protection against ground water contamination.

Major well design considerations for injection wells are (1) protection of underground sources of drinking water, (2) control of the corrosion of casings and cement, (3) the reduction of surface injection pressure, and (4) a reliable annulus monitoring system.

Specialized materials of construction are beginning to be used extensively in the waste disposal well industry to improve mechanical integrity, reduce workovers and most importantly, provide the best possible protection of fresh ground water resources.

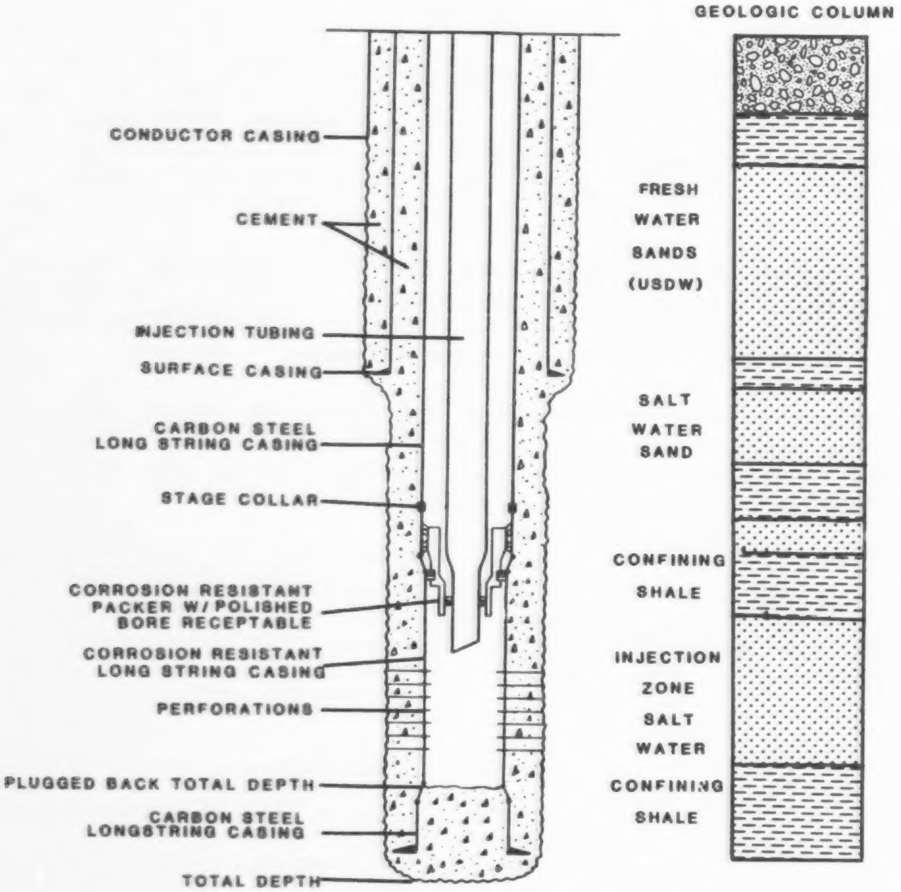
Recently installed injection wells utilize oversize corrosion resistant tubulars, optimized completions, acid resistant cements and specialized packer assemblies. Improved well construction results in reliable testing of mechanical integrity, a way to monitor the disposal well and provide preventative maintenance before ground water pollution occurs.

Introduction

The recently implemented Federal Underground Injection Control (UIC) program has established minimum design standards for Class I waste disposal wells to provide protection from ground water contamination. Technological advancements in the "state of art" construction techniques for Class I wells allow even greater protection of fresh ground water resources and result in a longer, safer operational life.

Many recently installed waste disposal wells utilize oversize corrosion resistant casings, fiberglass reinforced epoxy tubing, acid resistant resin cements and specialized packer assemblies. These specialized materials of construction incorporated into a properly designed disposal well can improve mechanical integrity, reduce workovers and provide the best possible protection of fresh ground water resources. A typical waste disposal well design with perforated completion is shown below in Figure 1.

FIGURE 1


GEOLOGIC COLUMN
CONDUCTOR CASING
CEMENT
INJECTION TUBING
SURFACE CASING
CARBON STEEL LONG STRING CASING
STAGE COLLAR
CORROSION RESISTANT PACKER W/POLISHED BORE RECEPTABLE
CORROSION RESISTANT LONG STRING CASING
PERFORATIONS
PLUGGED BACK TOTAL DEPTH
CARBON STEEL LONGSTRING CASING
TOTAL DEPTH
FRESH WATER SANDS (USDW)
SALT WATER SAND
CONFINING SHALE
INJECTION ZONE SALT WATER
CONFINING SHALE

CLASS 1 INJECTION WELL SCHEMATIC

Oversize Casing Design

Conventional well construction dictates that a disposal well be designed from the inside out to accomodate the required injection tubing. The concentric strings of casing are sized to incorporate an orderly sequence of steel and cement forming multiple layers of ground water protection. However, if the innermost protection casing is designed to be "one size oversize" the safe operational life of a waste disposal well can be extended in many situations. If the casing becomes damaged, the well can be relined not just "patched", retaining full mechanical integrity. This allows the well to still operate with same size injection tubing and therefore the same capacity.

Another benefit is that a larger injection tubing can be utilized initially resulting in reduced friction losses and lower wellhead pressures. Because a large portion of the permitted operating pressure is not used up just moving the wastewater down the tubing, the frequency of stimulations and/or workovers are minimized. Lower operating pressures improve mechanical integrity of all well components, reduce power requirements and keep maintenance costs down.

Oversize casing design provides waste disposal well operators with a relatively inexpensive insurance policy to recomplete their wells and a better way to maintain mechanical integrity over the operational life span.

Materials of Construction

Tubulars

The majority of the existing waste disposal wells have been constructed utilizing common API carbon steel casing. However, the demand for corrosion resistant completions to dispose of acidic wastestreams has brought about a change in the materials of construction for the protection casing. By combining carbon steel with corrosion resistant casing, the operational life of the protection casing can be significantly longer. Because only the casing below the packer is actually waste wetted, only a relatively short section of corrosion resistant casing is needed. The remaining protection casing can be API carbon steel.

The amount of corrosive resistant casing required varies on type of completion. Open hole completions require only approximately 100' immediately above the injection zone while perforated completions may need up to several hundred feet set through the injection zone (see Figure 1). Utilizing corrosion resistant casing results in only a small incremental increase in installed cost but yields a magnitude greater protection from corrosion.

Hastelloy C-276 is a commonly used corrosion resistant casing material. Its compatibility with many acids and high strength characteristics make Hastelloy C a preferred choice by the steel industries for use in the construction of waste pickle liquor injection wells. Even industrial brine wells (non-acidic) are also using stainless steels to reduce casing corrosion, provide packer protection and reduce workover costs.

Fiberglass reinforced epoxy (FRE) casing while being highly corrosion resistant is also very economical and widely available. However, it is significantly inferior to Hastelloy C in terms of resistance to mechanical damage. During installation, should any rock or other item get lodged between the borehole and the FRE casing, the FRE casing could be crushed. In many situations, there would be little or no indication of the damage at the surface resulting in failure of the cementing operations.

Even after FRE casing is successfully installed, it remains at risk due to damage from the packer, wireline operations, stimulation procedures or even high pressure mechanical integrity testing. Most new wells have not installed FRE casing but have chosen the higher strength exotic metals.

Epoxy Resin Cements

Because the greatest chance of upward migration of injected fluid is at the wellbore itself, the selection of a compatible cement is critical. This is especially true of acidic wastestreams where the common types of oilfield cements are unsuitable.

Epoxy resin cements originally developed for limited use in the oilfield are being employed frequently in the cementing of corrosion resistant disposal wells. Epoxy resin cement used in conjunction with corrosion resistant tubulars maintain the integrity of the confining layer and guard against ground water contamination. The epoxy cement should be emplaced a minimum of 300' to 500' above the top of the corrosion resistant casing. The epoxy cement forms a sheath of protection around the carbon steel casing, minimizing external corrosion through the confining layer.

Since the epoxy cement does not extend to the surface, a two stage cement job is required to allow for proper zone placement. In addition, two stage cementing reduces hydrostatic forces and common oilfield cements can be used for the remainder of the well. It is not economical or practical to use epoxy resin cements for the entire protection casing.

Injection Tubing

Fiberglass injection tubing has become the most commonly used tubing for acidic wastestream disposal.

It is highly corrosion resistant, exhibits lower friction loss and keeps waste fluids away from casing. It has a high capacity to expand or contract according to temperature variations. A tubing force calculation must be performed to evaluate packer - wellhead - tubing combination to correctly accomodate force balances.

Packers

Few will disagree that the packer is one of the most important elements in a waste disposal well. Its primary function is to form a hydraulic seal between the tubing and protection casing to allow continuous monitoring of well integrity. Packers are subject to very harsh environments and must be selected to be compatible with the pressures and wastestreams for a particular injection well.

Conventional oilfield packers present a number of well design and maintenance problems in some injection wells. Specifically, most affected are those wells requiring so called "exotic" materials of construction such as stainless steel, hastelloy or FRE for the protection string. It is common for "exotic" wells to use API carbon steel casings for the protection string down to just above the packer set point and then convert over to the "exotic" material for the remainder of the well. The recent development of a packer specifically designed for waste disposal wells has provided greater flexibility than conventional packers.

The following design features are incorporated:

- Packer slips divorced from the primary sealing elements.
- Packer slips located on top for setting in the carbon steel casing.
- Sealing elements on bottom of the packer are capable of being a different size than the slips so that the bottom nose of the packer can be inserted into reduced size exotic casings for a seal.
- Controlled setting pressures for the sealing elements to avoid damage to the exotic casings.

These basic design features provided a major breakthrough over conventional packers in case of the corrosion resistant injection wells. The advantages of the packer include:

- By designing the packer with slips on top, they can be set into the API carbon steel portion of the protection string.
- The exotic casing below can be isolated from all major packer forces resulting from setting the packer slips or jarring and milling operations during removal.
- Because the slips can be made of carbon steel, they are easily milled out and replaced if required.
- By having only the sealing elements of the packer extend down into the exotic casing, only the lowermost portion of the packer has to be constructed of exotic materials. Most key packer components are protected from corrosion behind the sealing element.

Annulus Monitoring System

The Federal UIC program mandates that all waste disposal wells be equipped with an annulus monitoring system to demonstrate that leakage of waste fluids is not occurring. Continuous monitoring is achieved by a simple but effective system. The wellhead, casing, tubing and packer form a sealed annulus which can be easily monitored. All key well components are checked with a single system.

The annulus monitoring system illustrated in Figure 2 has a pressure chamber located adjacent to the wellhead. This arrangement has the following advantages:

- Annulus remains full of fluid through a wide range of operating temperatures.
- Nitrogen pressure can be applied to vessel at wellhead.
- Loss of well integrity can be visually observed by both pressure and fluid level changes.

Basically, annulus monitoring systems can be operated in two different modes. The annulus pressure can either be maintained above or below the surface injection pressures. Table 1 illustrates each scenario as to how various well failures are detected by the monitoring system.

Table 1
Annulus Monitoring System

When Annulus Pressure is Below Surface Injection Pressure

Leak Site	Fluid Level	Annulus Pressure
Casing	Drop	Drop
Tubing	Rise	Equalizes with SIP
Packer	Rise/Drop Depending on SIP	Equalizes with SIP

When Annulus Pressure is Above Surface Injection Pressure

Leak Site	Fluid Level	Annulus Pressure
Casing	Drop	Drop
Tubing	Drop	Equalizes with SIP
Packer	Drop	Equalizes with SIP

SIP - Surface Injection Pressure

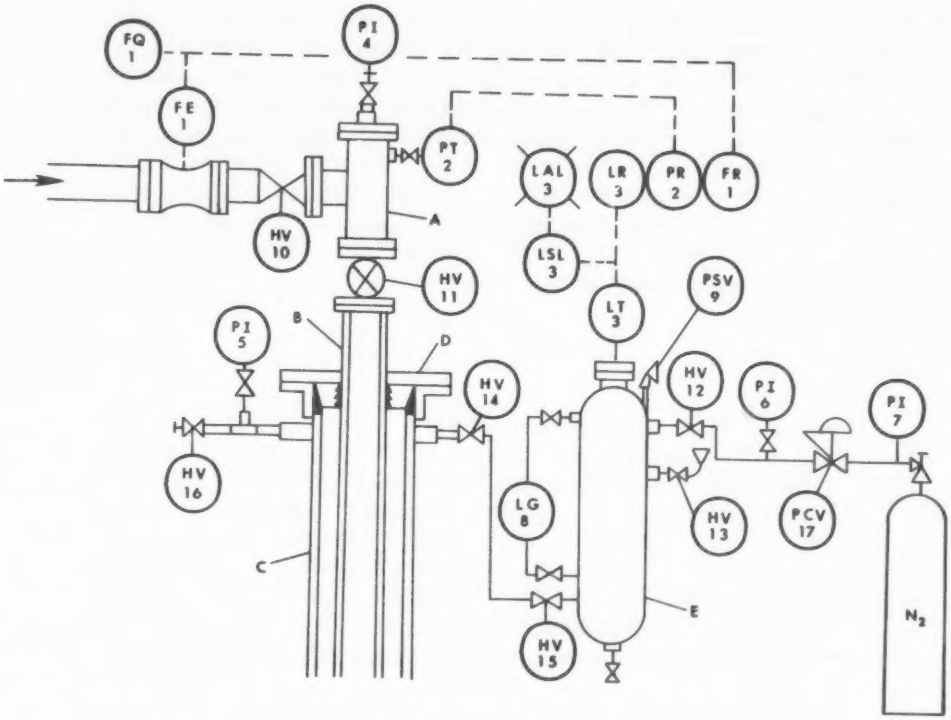


FIGURE 2

WELLHEAD AND ANNULUS MONITORING SCHEMATIC

A	Flow Tee
B	Tubing
C	Casing
D	Casing Head
E	Annulus Pressure Chamber
FE-1	Flow Sensing Element for Injected Fluid
FR-1	Flowrate Recorder
FQ-1	Flow Volume Totalizer for Injected Fluid
PT-2	Pressure Transmitter - wellhead tubing pressure
PR-2	Pressure Recorder - wellhead tubing pressure
LT-3	Level Transmitter - annulus pressure chamber level
LR-3	Level Recorder - annulus pressure chamber level
LSL-3	Low Level Switch - annulus pressure chamber level
LAL-3	Low Level Alarm - annulus pressure chamber level
PI-4	Pressure Gauge - wellhead tubing pressure
PI-5	Pressure Gauge - annulus pressure
PI-6	Pressure Gauge - annulus pressure chamber (during pressurized nitrogen)
PI-7	Pressure Gauge - nitrogen supply bottle
LG-8	Level Glass - annulus pressure chamber level
PSV-9	Pressure Relief Valve - annulus pressure chamber
HV-10	Wing Valve (supply valve) 2000 psi W.P.
HV-11	Master Valve 200 psi W.P.
HV-12	Nitrogen Supply Valve
HV-13	Annulus Pressure Chamber Fill Valve
HV-14	Annulus Pressure Valve (casing)
HV-15	Annulus Pressure Valve (pressure chamber)
HV-16	Annulus Fill/Bleed Valve
PCV-17	Nitrogen Pressure Control Valve

Summary

In an effort to optimize the protection of ground water resources, improved construction techniques and corrosion resistant materials of construction are being employed in recently installed Class I waste disposal wells. Advancements include utilizing oversize corrosion resistant tubulars, optimized completions, acid resistant cements and specialized packer assemblies. Proper well design coupled with wastestream compatible well components and a monitorable annulus system are integral to the safe long term operation of a waste disposal well. Technological advancements have improved mechanical integrity and provide greater protection of ground water resources.

Biographical Sketches

Thomas A. Jones, Vice President/Senior Geologist, Golden StrataServices, Inc., 1000 Louisiana, Suite 2000, Houston, Texas, (713)759-9764, has specialized in the complex field of environmental regulation and its application to waste disposal wells. After graduating from the University of Texas with a B.S. Degree in Geology, Tom gained valuable experience with the Texas Department of Water Resources (TDWR) Underground Injection Control Section. His responsibilities with Golden StrataServices started in 1980 and has since become involved with permitting and installation of waste disposal wells nationwide.

Jill S. Haimson, Geologist, Golden StrataServices, Inc., 1000 Louisiana, Suite 2000, Houston, Texas, (713)759-9764, has specialized in the preparation of detailed UIC permit applications throughout the United States. Jill previously worked with the Arizona Department of Water Resources where she participated in geohydrologic investigations, ground water flow modeling and provided assistance in state wide ground water management programs. After receiving a Bachelors Degree in Geology at Queens College (CUNY), she did some graduate work at Arizona State University. She is a member of the National Water Well Association.

Session III
Dealing with Mining and Mine Wastes

REGULATORY STRATEGY GOVERNING THE DISCHARGE OF MINING WASTE TO LAND IN CALIFORNIA

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Abstract

The California State Water Resources Control Board has recently adopted new regulations that apply to discharges at mining waste management units such as waste piles, surface impoundments, and tailings ponds. The regulations contain specific provisions for establishing categories of mining wastes based on an assessment of the potential risk of degrading water quality by each waste group. Depending on the degradation potential by each group, the new regulations specify siting criteria and standards for construction of mining waste management units for the purpose of averting failure of containment structures due to adverse impacts such as those associated with the occurrence of active faults, landslides, and land subsidence. Floodplain siting criteria and construction standards are also provided to ensure that mining waste management units are properly protected from to 100-year frequency flooding. Other standards for construction include those for the design and installation of clay and synthetic liners in combination with leachate collection and removal systems to prevent the movement of fluids, including waste and leachate through those containment structures. Criteria for the design of precipitation and drainage controls are included to accommodate location-specific precipitation events to effectively protect against failures such as waste overflows or washouts. Furthermore, the new regulations specify that mining waste management units be provided with surface and ground water monitoring systems to obtain an indication of the adequacy of containment structures or if additional requirements are necessary to protect water quality. Provisions for proper closure of new and existing mining waste management units are included to ensure the physical and hydraulic integrity of containment structures for as long as wastes pose a threat to water quality.

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Introduction

On May 1, 1984, the State Water Resources Control Board (State Board) repealed the existing land disposal regulations contained in Subchapter 15 of Chapter 3 of Title 23 of the California Administrative Code (CAC) and adopted new regulations governing discharges of hazardous and non-hazardous waste to land, to be codified in the same Subchapter 15. The new Subchapter 15 regulations, which primarily apply to classified waste management units, were developed concurrently with the revised hazardous waste management regulations that are in preparation by the California Department of Health Services (DHS).

Water quality degradation, that is a consequence of active and inactive mining operations, has been a major environmental issue in California for at least the last three decades. Such degradation is attributable to mine discharges that cause adverse chemical and physical impacts on water quality mainly in the form of acidification, pollution by metals, and sedimentation. Acidification commonly is due to the reaction which entails sulfide oxidation and decomposition in the presence of water. Consequent metallic and nonmetallic ion concentrations that can occur in mine drainage can be extremely damaging to aquatic life and wildlife habitat. Adverse impacts due to sedimentation or siltation include the filling of stream beds which impair aquatic life and water supply facilities.

Acknowledging the uniqueness of waste discharge problems associated with mining activities, the State Board has provided for the regulation of such discharges under separate provisions contained in Article 7 of the new subchapter. The provisions establish discharge requirements for mining waste management units such as waste piles, surface impoundments or tailings ponds, and related treatment or storage facilities.

The nine California Regional Water Quality Control Boards (Regional Boards) have the authority to regulate mining-related discharges by issuing waste discharge requirements. Such requirements may include discharge prohibitions in compliance with water quality objectives in applicable Water Quality Control Plans that are regionally established throughout the State.

Previous Subchapter 15 regulations applied to discharges of mining waste. However, those did not provide specific provisions regarding the siting and development of suitable disposal facilities to properly contain such discharges. The new regulations provide that guidance to Regional Boards as well as to dischargers in the attempt to achieve complete waste containment to protect against adverse water quality impacts in conformance with the State Board's nondegradation policy. That policy states that whenever the existing quality of water is better than the quality in adopted water quality control policies, such existing high quality will be maintained until it has been demonstrated to the State that any change will be consistent with maximum benefit to the people of the State, will not unreasonably affect present and anticipated beneficial use of such water, and will not result in water quality less than that prescribed in the policies.

Applicability

Mining waste management units which receive waste discharge requirements after the effective date of the subchapter are subject to or must comply with requirements as summarized in Table 1. Regional Boards will determine the extent to which existing active and inactive mining waste management units shall comply with those siting and construction requirements. Upon submittal of a complete report of waste discharge for any unit, the appropriate Regional Board will prepare and adopt waste discharge requirements which implement the necessary provisions of Article 7 or may impose more stringent requirements depending on the regional and location-specific conditions. On the other hand, the Regional Board could waive the adoption of waste discharge requirements if it finds that water quality degradation will not occur due to the mining-related activities.

Mining Waste Groups

In the context of the regulations, mining waste is that generated from the mining and processing of ores and mineral commodities. Such mining waste is considered to include: 1) overburden, usually consisting of a soil and rock mixture which has been removed from its natural site or relocated; (2) waste rock which has been removed or relocated but has not been processed; and (3) solid residues, sludges, and liquids from the processing of ores and mineral commodities.

Adverse impacts on water quality is dependent primarily on the chemical and physical characteristics of mining wastes. In conformance with Article 7, such wastes can be assigned by the Regional Board as Group A, Group B, or Group C, based on an assessment of the potential risk presented by each category to degrade water quality. The criteria for group assignment is as follows:

1. Group A mining wastes are those that must be managed as hazardous waste pursuant to DHS regulations (see Section 66300 of Title 22 of the CAC) provided that the Regional Board finds that such mining wastes pose a significant threat to water quality.

2. Group B wastes consist of or contain nonhazardous soluble pollutants in concentrations which exceed regional or local water quality objectives and which could cause water quality degradation. Group B wastes also include those that consist of or contain "manageable" hazardous wastes (granted a variance from hazardous waste management requirements pursuant to Section 66310 of Title 22 of the CAC), provided that the Regional Board finds that such mining wastes pose a low-risk potential for degrading water quality.

3. Group C mining wastes are relatively inert wastes from which any discharge would be in compliance with regional or local water quality objectives, other than turbidity, and with the State Board's nondegradation policy.

TABLE 1

Summary of Requirements for New and Existing Mining Waste Management Units

Type of Requirement	New Units	Existing Units	Exemptions
Siting	Locate away from Holocene faults; locate outside of areas of rapid geologic change; provide flood protection as required by regional boards	Peak streamflow protection as required by regional boards	New units may be sited in areas of rapid geologic change if containment structures are designed and constructed to preclude failure
Construction	Liners of low permeability natural geologic materials leachate collection and removal system for Groups A and B units; precipitation and drainage controls	Precipitation and drainage controls	No liners of collection and removal systems required for Group C units; new waste piles may be exempted from these systems if it can be demonstrated that leachate will not form or escape; contingency plan required; additional monitoring may be required
Monitoring	Ground water; surface water; unsaturated zone, as feasible		None
Closure and Post-Closure Maintenance	Closed and maintained to mitigate threat to water quality		None

Group B mining wastes are generally those that are soluble in water 5
but which are unlikely to affect the quality of surface and ground water as adversely as those in Group A. Group C mining wastes are those that are nonwater soluble but which can cause turbidity and sedimentation in surface water when such wastes are improperly discharged or contained.

It is improper from a water quality stand point to categorically establish groups of mining wastes on the basis of unprocessed (overburden and waste rock) and processed rock with the associated waste materials. In some cases, overburden or unprocessed waste rock may present as great or greater risk to water quality than that posed by processed rock. To categorize a mining waste as either Group B or Group C, the Regional Board may consider factors such as: (1) possible low-concentrations of hazardous constituents in the waste, (2) possible low acid-production potential of the waste, and (3) the intrinsic properties of the waste that may facilitate the desired containment. Furthermore, the Regional Board may require that a mining waste be treated or neutralized whenever feasible to minimize the threat to water quality and possibly to achieve containment more readily.

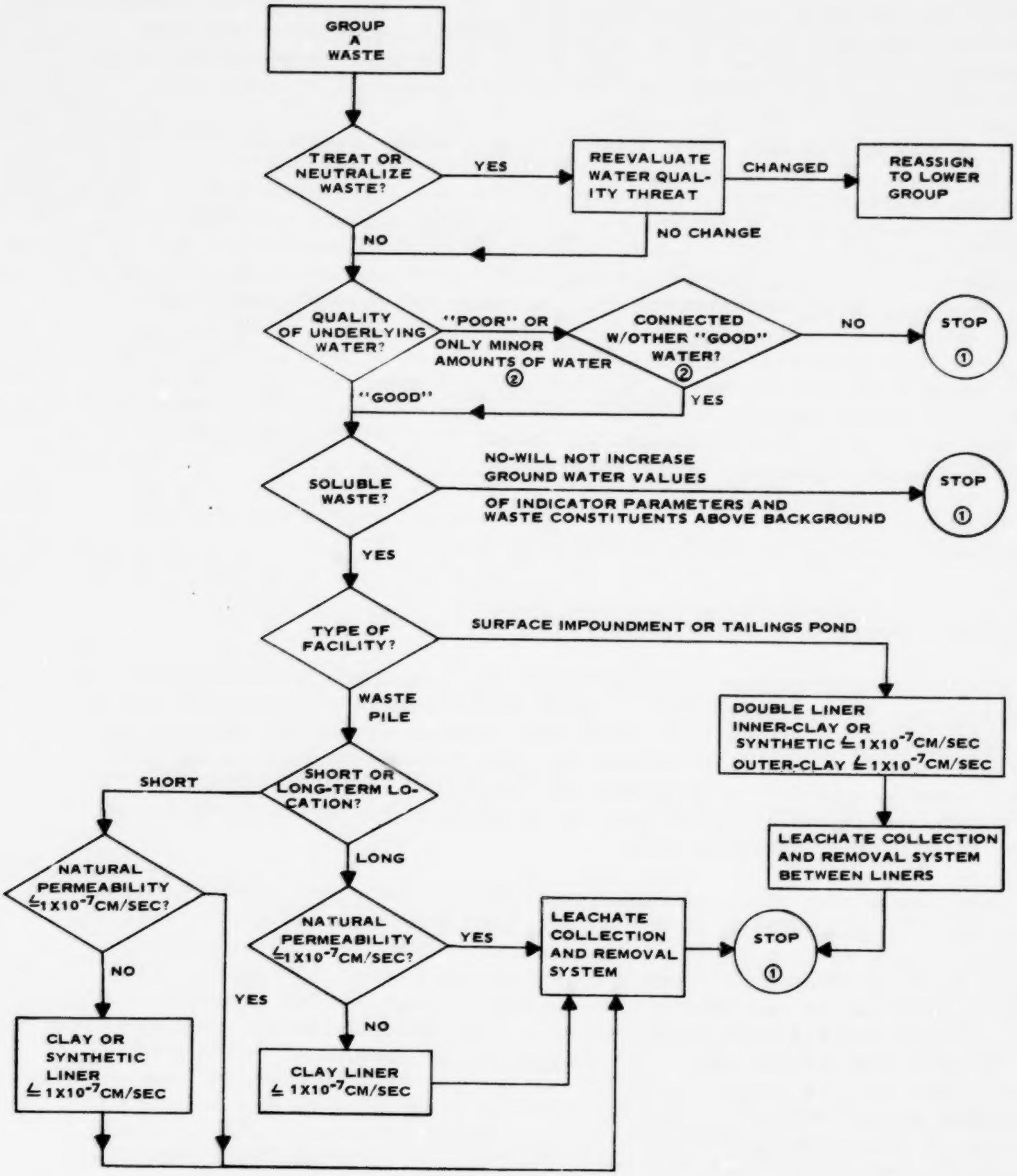
Procedures for determining discharge alternatives for Group A and Group B mining wates are shown in Figures 1 and 2.

Mining Waste Management Unit Siting Criteria

Mining operations are seldom located where geologic and hydrologic conditions are naturally optimal to effectively preclude ground and surface water quality degradation. Therefore, mining wastes are subject to regulation under separate provisions not only because on-site conditions may not usually qualify for approval as classified waste management units, but also in recognition that it is impractical to transport mining wastes to distant classified units.

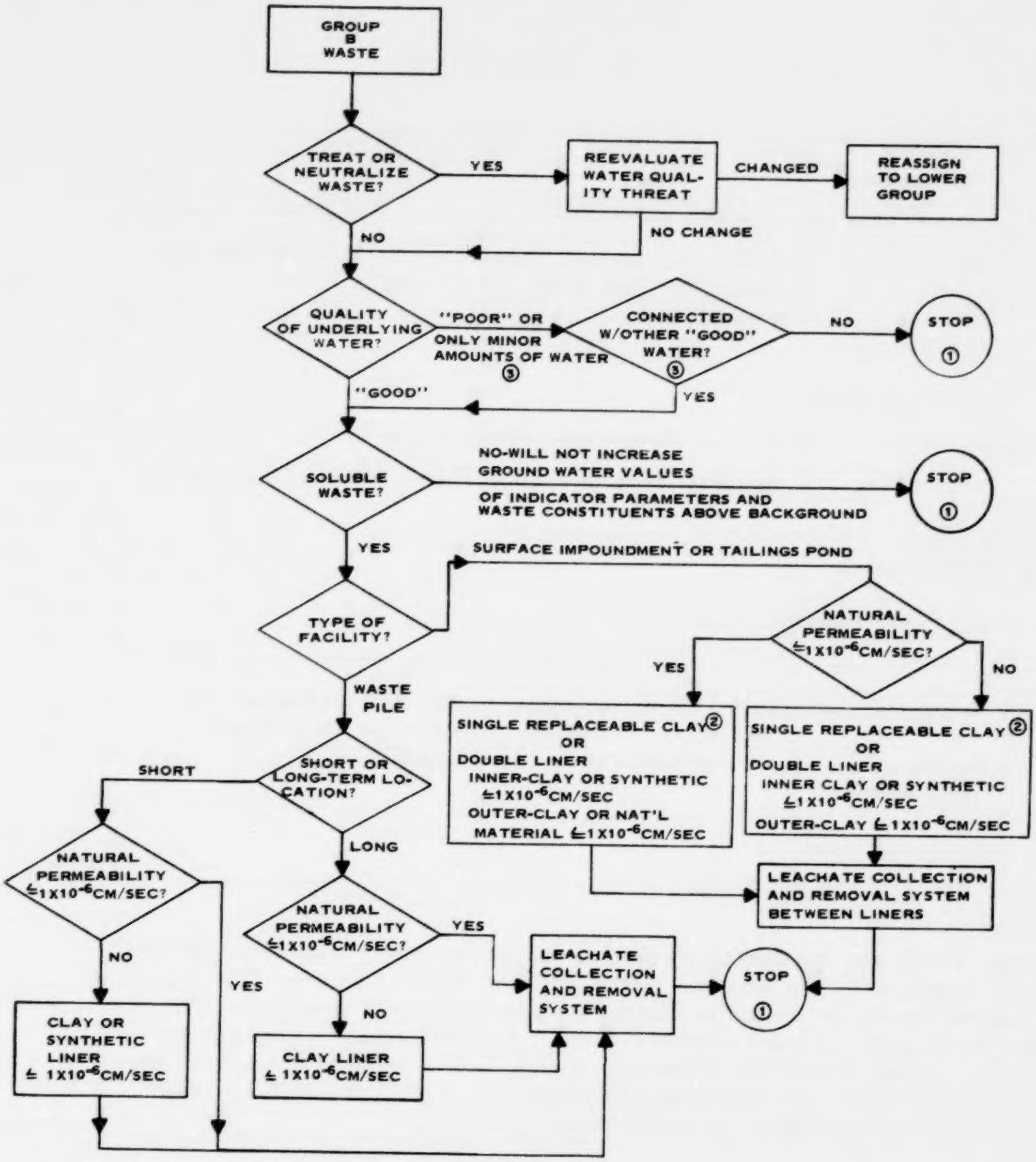
Only new mining waste management units must meet two geologic siting criteria. For the purpose of minimizing the rupture potential of containment structures at Group A and Group B units, there is a criterion that no portion of a Group A or Group B unit overlie a known Holocene fault (a fault which is or has been active during the last 11,000 years) to negate direct fault displacement impacts. The other criterion requires that Group A and Group B units not be subject to impacts of rapid geologic change such as those caused by landslides, land subsidence, and associated ground movement. New Group A and Group B units may be sited in areas of rapid geologic change if containment structures are designed and constructed to preclude failure. New Group C units may be located on Holocene faults if direct displacement will not cause escape of wastes or irreparable damage to containment structures.

The floodplain siting criterion for new Group A mining waste management units and peak streamflow requirements for all units is as shown in Table 2. New waste management units are to be sited for the purpose of preventing inundation, washout, and consequent spreading of waste due to 100-year frequency flooding. Existing Group A and Group B units that are


GROUP A WASTE
TREAT OR NEUTRALIZE WASTE?
YES
REEVALUATE WATER QUALITY THREAT
CHANGED
REASSIGN TO LOWER GROUP
NO CHANGE
NO
QUALITY OF UNDERLYING WATER?
"POOR" OR ONLY MINOR AMOUNTS OF WATER ②
CONNECTED W/OTHER "GOOD" WATER? ②
NO
STOP ①
YES
"GOOD"
SOLUBLE WASTE?
NO-WILL NOT INCREASE GROUND WATER VALUES OF INDICATOR PARAMETERS AND WASTE CONSTITUENTS ABOVE BACKGROUND
STOP ①
YES
TYPE OF FACILITY?
SURFACE IMPOUNDMENT OR TAILINGS POND
DOUBLE LINER INNER-CLAY OR SYNTHETIC ≤ 1X10^-7 CM/SEC OUTER-CLAY ≤ 1X10^-7 CM/SEC
LEACHATE COLLECTION AND REMOVAL SYSTEM BETWEEN LINERS
WASTE PILE
SHORT OR LONG-TERM LOCATION?
SHORT
LONG
NATURAL PERMEABILITY ≤ 1X10^-7 CM/SEC?
NO
YES
CLAY OR SYNTHETIC LINER ≤ 1X10^-7 CM/SEC
NATURAL PERMEABILITY ≤ 1X10^-7 CM/SEC?
YES
NO
CLAY LINER ≤ 1X10^-7 CM/SEC
LEACHATE COLLECTION AND REMOVAL SYSTEM
STOP ①

① Containment features to protect surface water are required.
② Determination pursuant to comprehensive hydrogeologic investigation.

FIGURE 1. Disposal alternatives for Group A wastes


GROUP B WASTE
TREAT OR NEUTRALIZE WASTE?
YES
REEVALUATE WATER QUALITY THREAT
CHANGED
REASSIGN TO LOWER GROUP
NO
NO CHANGE
QUALITY OF UNDERLYING WATER?
"POOR" OR ONLY MINOR AMOUNTS OF WATER ③
CONNECTED W/OTHER "GOOD" WATER? ③
NO
STOP ①
"GOOD"
YES
SOLUBLE WASTE?
NO-WILL NOT INCREASE GROUND WATER VALUES OF INDICATOR PARAMETERS AND WASTE CONSTITUENTS ABOVE BACKGROUND
STOP ①
YES
TYPE OF FACILITY?
SURFACE IMPOUNDMENT OR TAILINGS POND
NATURAL PERMEABILITY ≦1X10⁻⁶CM/SEC?
YES
NO
SINGLE REPLACEABLE CLAY② OR DOUBLE LINER INNER-CLAY OR SYNTHETIC ≦1X10⁻⁶CM/SEC OUTER-CLAY OR NAT'L MATERIAL ≦1X10⁻⁶CM/SEC
SINGLE REPLACEABLE CLAY② OR DOUBLE LINER INNER CLAY OR SYNTHETIC ≦1X10⁻⁶CM/SEC OUTER-CLAY ≦1X10⁻⁶CM/SEC
LEACHATE COLLECTION AND REMOVAL SYSTEM BETWEEN LINERS
WASTE PILE
SHORT OR LONG-TERM LOCATION?
SHORT
LONG
NATURAL PERMEABILITY ≦1X10⁻⁶CM/SEC?
NO
YES
CLAY OR SYNTHETIC LINER ≦ 1X10⁻⁶CM/SEC
NATURAL PERMEABILITY ≦1X10⁻⁶CM/SEC?
YES
NO
CLAY LINER ≦ 1X10⁻⁶CM/SEC
LEACHATE COLLECTION AND REMOVAL SYSTEM
STOP ①

①Containment features to protect surface water are required.
②Surface impoundment only.
③Determination pursuant to comprehensive hydrogeologic investigation.

FIGURE 2. Disposal alternatives for Group B wastes

Table 2

Floodplain Requirements

Waste Group	Waste Management Unit	Existing Units	New Units
A	Waste Pile Surface Impoundment Tailings Pond	Protect from not less than 100-year peak streamflow	Locate outside 100-year floodplain
B	Waste Pile Surface Impoundment Tailings Pond	Protect from not less than 100-year streamflow	
C	Waste Pile Surface Impoundment Tailings Pond	Retrofit to prevent sediment increases in surface water	Prevent sediment increases in surface water

improperly sited may be retrofitted for the necessary protection against potential flooding. New Group C units are not to be located within perennial, intermittent, or ephemeral stream channels unless controls are provided to properly divert surface runoff from such waste containment facilities. Group C wastes generated during seasonal mining operations may be exempted from the requirements provided that increased sediment discharges to surface water is prevented.

Understanding that siting considerations apply only to new mining waste management units, it is anticipated that implementation of corresponding requirements will enhance the long-term structural integrity of containment features and appurtenances by virtue of negating adverse impacts due to geologic and flooding hazards.

Mining Waste Management Unit Construction Standards

The natural and artificial containment feature requirements in Table 3 reflect the limited selectivity of waste management unit locations. Principle differences from the classified waste management unit requirements are that: (1) natural low permeability geologic materials are not required, (2) there are no synthetic liner requirements for Group A units to comply with hazardous waste management performance standards, and (3) leachate collection and removal systems for tailings ponds must be designed to withstand the ultimate weight of fluids, including waste and leachate, and solid or semi-solid wastes.

Table 3

Natural and Artificial Containment Features for Mining Waste Management Units

Waste Group	Waste Management Unit	Natural Permeability (cm/sec)	Liner(s) (permeability values in cm/sec)	Leachate Collection and Removal System
A	Waste Pile	$\leq 1x10^{-7}$	OR single clay liner $\leq 1x10^{-7}$	Required
	Surface Impoundment or Tailings Pond	Not applicable	Double liner, both $\leq 1x10^{-7}$, outer liner: clay; inner liner: clay or synthetic	Required
B	Waste Pile	$\leq 1x10^{-6}$	OR single clay liner $\leq 1x10^{-6}$	Required
	Surface Impoundment or Tailings Pond	Not Applicable	Double liner, both $\leq 1x10^{-6}$, outer liner: clay or natural permeability; inner liner: clay or synthetic; or replaceable clay liner	Required
C	Waste Pile, Surface Impoundment or Tailings Pond	Not applicable.		

As indicated in Table 3, the $1\text{x}10^{-7}$ cm/sec or less natural permeability of in-place geologic materials or for artificial liners to be used for Group A waste containment was selected because it is within a range that generally is considered to be relatively impervious. Permeabilities of no more than $1\text{x}10^{-7}$ cm/sec which can be reliably and consistently measured are required to be relative to the fluids, including waste and leachate, to be contained. Because of the lesser water quality degradation potential, the maximum allowable $1\text{x}10^{-6}$ cm/sec natural or artificial permeability was selected to contain Group B mining wastes.

Where permeabilities of natural in-place geologic materials are unsuitable for waste containment at new Group A and Group B waste pile unit locations, it is required that a minimum 2-foot thick clay liner be installed at a relative compaction of at least 90 percent. A "clay liner" in the requirements refers to a mixture of clay and other suitable fine-grained materials which can be compacted to attain and sustain the required permeability when installed. Furthermore, a synthetic liner of a minimum 40-mil thickness may be substituted for a clay liner at Group A and Group B waste piles if such liners will be used for less than 5 years.

The reason for limiting the use of liners to 5 years is that there are no known instances in California where synthetic liners have effectively contained wastes for more than 1 to 2 years. Nor have there been any field studies which demonstrate that synthetic liners are able to function without failure for more than 5 years. Synthetic liner failure usually has been caused by tearing and detrioration. In comparison, properly-installed clay liner systems appear to retain the desired physical and hydraulic characteristics for longer periods.

If the Regional Board exempts a discharger or operator from liner requirements at a waste pile, a contingency plan for alternative containment must be prepared. This plan is to be implemented if the alternative containment plan fails.

A permeability of $1\text{x}10^{-7}$ cm/sec or less is required for both liners of a double-lined Group A surface impoundment or tailings pond because it is anticipated that the hydraulic head buildup and related waste load pressures will eventually cause the inner liner to leak. As indicated in Table 3, the outer liner at these Group A units must consist of artificially-installed clay and the inner liner may consist either of artificially-installed clay or synthetic materials. Corresponding Group B units must be provided with double liners each with a permeability of no more than $1\text{x}10^{-6}$ cm/sec. The outer liner for these Group B units may consist either of artificially-installed clay or of $1\text{x}10^{-6}$ cm/sec in-place natural geologic materials. Inner liners for double-lined Group B units may consist of artificially-installed clay or synthetic materials.

A single clay liner for a Group B surface impoundment or tailings pond may be substituted for the double-lined system. However, the single clay liners must be removed or replaced before the outermost 25 percent (minimum of 1-foot thickness) is penetrated by fluid, including waste and leachate.

The leachate collection and removal systems required for Group A and Group B mining waste management units must be of the blanket type and be able to withstand the ultimate weight of fluids, including waste and leachate, and solid or semi-solid wastes. This is to ensure that there will not be significant hydraulic head buildup on an underlying liner to minimize the potential for liner penetration by wastes and leachate.

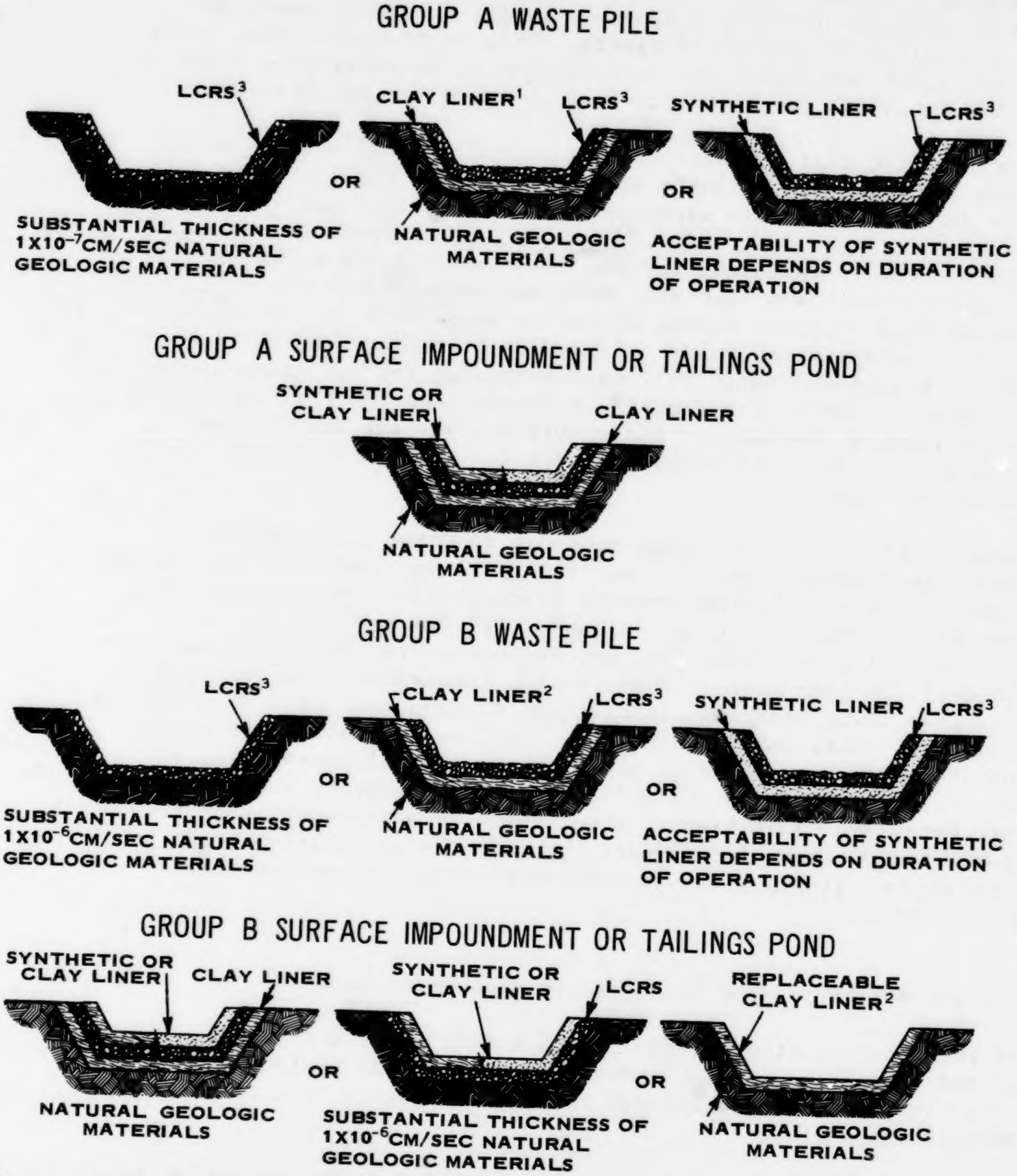
It is required that leachate collection and removal systems be installed immediately above a single liner, or between the inner and outer liner of a double-lined waste management unit. Leachate collection and removal systems must be designed, constructed, maintained, and operated to collect and remove twice the maximum anticipated daily volume of fluid, including waste and leachate, that may emanate from the unit. This is to provide a safety factor of two to compensate for the uncertainties in calculating the volume of waste and leachate that will be generated. The Regional Board will specify design and operating conditions in waste discharge requirements to ensure that there is no buildup of hydraulic head on the liner. Acceptable designs for the various natural and artificial containment features for Group A and Group B units are depicted in Figure 3.

The precipitation and drainage controls at Group A and Group B mining waste management units are required to be designed and constructed to accommodate the anticipated on-site precipitation and corresponding peak flows from surface runoff as follows: (1) Group A--one 25-year, 24-hour storm; (2) Group B-one 10-year, 24-hour storm; and (3) Group C--one 10-year, 24-hour storm. These design requirements are for the purpose of preventing washouts, inundation, erosion, and related potential failures of containment structures. Sizing is to allow for proper carryover storage capacity and to negate the potential overtopping of wastes due to direct precipitation for the given frequency storm event for each waste management unit group. Also, there is a minimum 2-foot freeboard special requirement for surface impoundments or tailings ponds to further prevent overtopping due to adverse wind conditions and corresponding wave action.

Water Quality Monitoring for Mining Waste Management Units

New and existing Group A and Group B mining waste management units are required to develop and implement ground water and surface water monitoring plans. Also, unsaturated zone monitoring is required at these units where feasible.

Required monitoring includes instituting a detection monitoring program, the details of which will be prescribed by the Regional Board. If water quality degradation is detected, the discharger or operator is to formulate and implement a detailed monitoring program to verify the existence and extent of water quality degradation caused by the mining-related activities. Depending on the verification monitoring findings, the operator may be required to prepare and implement a corrective action program that must be approved by the Regional Board. The intent of the



GROUP A WASTE PILE

LCRS[3]
OR
CLAY LINER[1]
LCRS[3]
OR
SYNTHETIC LINER
LCRS[3]
SUBSTANTIAL THICKNESS OF 1X10^{-7}CM/SEC NATURAL GEOLOGIC MATERIALS
NATURAL GEOLOGIC MATERIALS
ACCEPTABILITY OF SYNTHETIC LINER DEPENDS ON DURATION OF OPERATION

GROUP A SURFACE IMPOUNDMENT OR TAILINGS POND

SYNTHETIC OR CLAY LINER
CLAY LINER
NATURAL GEOLOGIC MATERIALS

GROUP B WASTE PILE

LCRS[3]
OR
CLAY LINER[2]
LCRS[3]
OR
SYNTHETIC LINER
LCRS[3]
SUBSTANTIAL THICKNESS OF 1X10^{-6}CM/SEC NATURAL GEOLOGIC MATERIALS
NATURAL GEOLOGIC MATERIALS
ACCEPTABILITY OF SYNTHETIC LINER DEPENDS ON DURATION OF OPERATION

GROUP B SURFACE IMPOUNDMENT OR TAILINGS POND

SYNTHETIC OR CLAY LINER
CLAY LINER
OR
SYNTHETIC OR CLAY LINER
LCRS
OR
REPLACEABLE CLAY LINER[2]
NATURAL GEOLOGIC MATERIALS
SUBSTANTIAL THICKNESS OF 1X10^{-6}CM/SEC NATURAL GEOLOGIC MATERIALS
NATURAL GEOLOGIC MATERIALS

[1] LINER PERMEABILITIES ARE 1X10^{-7}CM/SEC OR LESS

[2] LINER PERMEABILITIES ARE 1X10^{-6}CM/SEC OR LESS

[3] AS REQUIRED BY THE REGIONAL BOARD

FIGURE 3. Summary of liner requirements for Group A and B waste piles and surface impoundments or tailings ponds

monitoring programs is to establish a prompt indication of the adequacy of the siting criteria and construction standards, or if additional requirements, which may possibly include unit closure, are necessary to protect water quality.

The provisions for exemptions from certain construction standards for some Group A and Group B mining waste piles is based on the assumption that fluid, including waste and leachate, cannot be produced due to certain climatic conditions that occur in arid and semi-arid areas. Factors underlying this assumption would include the cumulative effects of discharging low-moisture content materials at mining waste piles in combination with the absence of significant precipitation, and the installation of artificial protective measures to prevent entrance of moisture from other sources. Although it is likely that the migration of waste and leachate from waste piles can be minimized under those special circumstances, a construction standard exemption dictates that an extensive monitoring plan be prepared and implemented at the waste pile unit to ensure that total containment is, in fact, achieved as long as such wastes pose a threat to water quality. Such assurance is necessary to compensate for the absence of the usual containment structures.

Closure and Post-closure Maintenance of Mining Waste Management Units

New and existing mining waste management units must be closed properly to ensure the long-term physical and hydraulic integrity of containment structures and to avert improper post-closure activities that could impair the integrity of necessary containment features and cause adverse impacts on water quality. To accomplish this, the dischargers must develop and submit closure and post-closure maintenance plans for Regional Board approval. These plans are to include provisions ensuring compliance with applicable waste containment standards and monitoring program requirements for the purpose of preventing water quality degradation during the post-closure maintenance period.

Assurances that the discharges is financially capable of implementing closure and post-closure maintenance plans must be provided to the Regional Board. This is to establish the financial means necessary to ensure that water quality degradation will be prevented after termination of the revenue-producing active period of the mining operations.

Summary

The State Board's new regulations governing waste discharges to classified waste management units include specific provisions governing discharges of mining waste at mine sites. Such provisions or guidance did not exist in previous Subchapter 15 regulations.

Each waste stream from mining operations will be categorized or grouped in accordance to the potential risk of degrading water quality and managed accordingly. On-site waste management units must be located away from active faults, protected from other geologic hazards, and protected against damage by 100-year frequency flooding. Depending on the water quality degradation potential and the waste to be discharged, the new regulations further specify appropriate containment structures, precipitation and drainage controls, monitoring programs, and closure and post-closure maintenance requirements for mining waste management units.

The new regulations will impose substantially increased siting criteria, construction standards, and monitoring requirements on mining operations.

Biography of Presenting Author

Mr. Torres is a Senior Engineering Geologist with 25 years experience in the performance of hydrogeologic studies principally associated with ground water quality degradation problems. He received a B.S. degree in geology from the University of Texas with graduate work in civil engineering. As a State Water Resources Control Board staff geologist for the last 13 years, he has provided hydrogeologic and geotechnical consulting services to the nine Regional Water Quality Control Boards in regard to the waste discharge permitting system and has been a consultant to other state agencies on ground and surface water quality matters.

EFFECTS OF URANIUM MILL TAILINGS ON GROUND WATER QUALITY: A HISTORICAL PERSPECTIVE

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Abstract

Since the early 1940's, commercial mills throughout the United States have processed uranium ores. The most voluminous wastes of the uranium milling process are the tailings and liquors used to slurry these tailings into disposal impoundments. Concerns about environmental quality and public health prompted regulatory agencies and mill operators to assess the effects of tailings disposal on surface water and ground water quality. Based on analyses of the distribution of aqueous contaminants at active and inactive tailings impoundments, the authors classified contaminant species into general categories depending on their abundance in the tailings and their mobility in hydrogeologic systems. Class I constituents are conservative species such as sulfate, nitrate, and chloride that are generally abundant in tailings and migrate at or about the same velocity as ground water flow. These constituents have been recognized for years as effective indicators of contaminant migration. Class II constituents include uranium, molybdenum, selenium, arsenic, and vanadium, which are abundant in tailings and relatively mobile in hydrogeologic systems. Although migration of these constituents is retarded relative to ground water flow, significant contaminant concentrations are observed distant from disposal impoundments. Class III constituents include other heavy metals and chemical constituents, as well as radionuclides (excluding uranium). Despite their abundance in tailings, these constituents are not observed above background concentrations at appreciable distances from tailings impoundments. Exceptions to this constituent classification are recognized and necessitate site-specific assessments of contaminant transport and water quality impacts. The classification supports development of ground water monitoring and corrective action programs. Historical perspectives about the mobility, transport, and fate of toxic constituents present in uranium mill tailings and their effects on ground water quality are essential to effective waste management as environmental strategies shift emphasis from remedial to protective actions.

Introduction

Since the dawn of the Atomic Age in the early 1940's, uranium mills have discharged large amounts of mill tailings into surface disposal impoundments. Seepage of mill tailings solutions from these impoundments into shallow aquifers has affected the quality of ground water at mill sites by increasing the concentrations of constituents above background levels. Review of the history of mill tailings management and evaluation of water quality data at conventional uranium mills provides a historical perspective about the effects of uranium mill tailings on ground water quality.

After a brief description of conventional uranium milling and mill tailings, this paper summarizes the history of mill tailings management in the United States and evaluates the effects of uranium mill tailings on ground water quality. A classification of ground water contaminants is introduced to aid in the generic discussion of the effects of mill tailings on ground water quality. By examining the historical management of mill tailings and the effects of this management on ground water quality, present and future tailings management may be modified to protect ground water.

Uranium Milling and Mill Tailings

Sandstone uranium deposits, the prinicipal source of uranium in the United States, contain a variety of metals including arsenic, molybdenum, selenium, and vanadium (Rose, Hawkes, and Webb, 1979). These metals are generally soluble under oxidizing conditions and insoluble under reducing conditions. During ore genesis, oxidizing ground water transports dissolved metals into reducing zones where the metals are precipitated. Continual influx of uranium-bearing ground water into reducing zones may precipitate economic deposits of uranium.

Conventional uranium milling chemically reverses the ore genesis process by oxidizing and solubilizing the metals. In the milling process, uranium ore is leached by sulfuric acid in acid-leach circuits or by sodium carbonate-bicarbonate solutions in alkaline-leach circuits. Most (90%) operating conventional mills in the United States recover uranium by using an acid-leach process (White, 1984). As shown in Figure 1, sodium chlorate and other oxidants are typically added in the leach circuits to solubilize uranium.

After the leaching circuit, the uranium-bearing solution is separated from the ore sediments (sands and slimes) that once contained the uranium. The dilute uranium solution is then concentrated and purified in solvent extraction or ion exchange circuits, where the uranium is removed to a non-aqueous phase (i.e., into the solvent or onto ion exchange resins). Organic compounds, such as kerosene (solvent) and tertiary amines (concentrating agents), are introduced into the milling process in solvent extraction circuits.

Concentrated uranium solutions are then eluted from "loaded" solvent or anionic resin. In some alkaline-leach mills, the concentration process is simplified by precipitating uranium directly from the leach solution. Following the concentration circuit, uranium compounds are precipitated from concentrated aqueous solutions and prepared as yellowcake, a semi-refined uranium product.

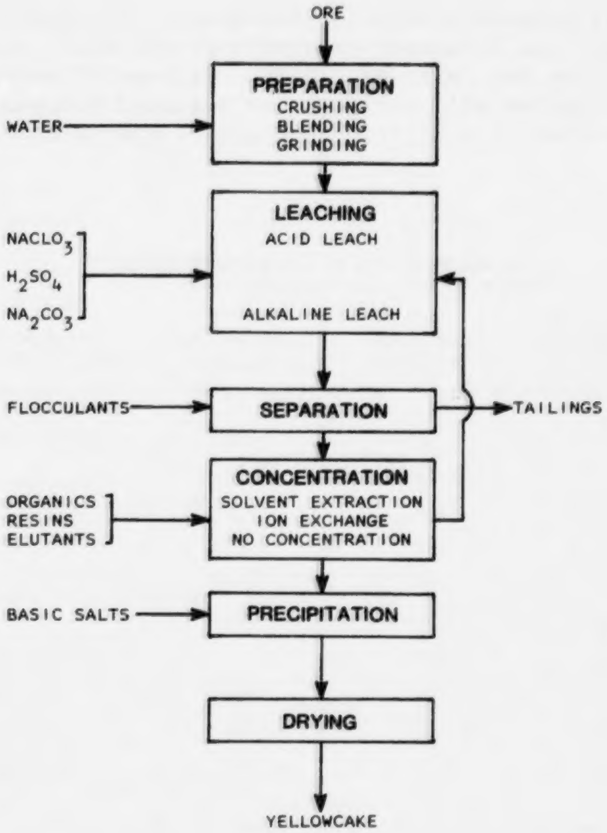
ORE
PREPARATION
CRUSHING
BLENDING
GRINDING
WATER
LEACHING
ACID LEACH
ALKALINE LEACH
NACLO$_3$
H$_2$SO$_4$
NA$_2$CO$_3$
FLOCCULANTS
SEPARATION
TAILINGS
CONCENTRATION
SOLVENT EXTRACTION
ION EXCHANGE
NO CONCENTRATION
ORGANICS
RESINS
ELUTANTS
BASIC SALTS
PRECIPITATION
DRYING
YELLOWCAKE

Figure 1. Conventional uranium milling process.

Uranium mill tailings consist of solid and liquid wastes generated by the extraction of uranium from ores milled primarily for their uranium values. Mill tailings wastes include leached ore solids, process chemicals, solubilized ore constituents not retained by the milling process, and reaction products that form during milling. As shown in Table I, mill tailings solutions typically contain elevated concentrations of soluble salts, radionuclides, heavy metals, and transition metals. Many of the heavy metals, as noted previously, exist at elevated concentrations in uranium ores and are solubilized along with the uranium by the milling process. Major cations and anions are added to tailings solutions by reactions of milling reagents with the ore. Uranium mill tailings solutions are characterized by high oxidation potentials, high ionic strengths, high concentrations of soluble salts, and high or low pH's depending upon the leach process.

After they are separated from uranium-bearing leach solutions, uranium mill tailings are slurried as solid-liquid mixtures (1:1 by weight) into surface disposal impoundments. Seepage of mill tailings solutions may have accounted for more than 80% of the liquid loss from early tailings impoundments (Merritt, 1971). Most of these early impoundments are unlined and

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constructed on relatively permeable alluvial sediments. Prolonged seepage from tailings impoundments has developed extensive ground water mounds at several mill sites (Williams and Osiensky, 1983). Seepage of concentrated tailings solutions from uranium mill tailings has affected the quality of shallow ground water in alluvial aquifers downgradient hydraulically from tailings impoundments.

TABLE 1. DISSOLVED CONSTITUENT CONCENTRATIONS(MG/L) IN URANIUM MILL TAILINGS SOLUTIONS

CONSTITUENT/ PARAMETER	MAXIMUM REPORTED CONCENTRATION[1]	ACID LEACH TAILINGS[2]	ALKALINE LEACH TAILINGS[3]	DRINKING WATER QUALITY CRITERIA[4]
URANIUM	250	40.0	26.10	0.035 (N)
ARSENIC	42	3.5	2.46	0.05 (P)
MOLYBDENUM	93.7	0.35	76.57	---
SELENIUM	36.7	0.6	5.0	0.01 (P)
LEAD	2.61	---	0.47	0.05 (P)
TH-230(PCI/L)	250,000	---	---	---
RA-226(PCI/L)	35,000	---	---	5 (P)
IRON	5,450	2,215	---	0.3 (S)
CHLORIDE	2,400	97	2092	250 (S)
SULFATE	100,000	12,850	14,540	250 (S)
TDS	150,000	---	29,656	500 (S)

REF: (1) BASED ON EPA, 1983, AND WEBER, 1983; (2) SERNE, PETERSON, AND GEE, 1983; (3) LONGMIRE, 1983; AND (4) (P) 40 CFR 141, (S) 40 CFR 143, AND (N) NRC, 1983.

Historical Management of Uranium Mill Tailings

The uranium milling industry began in the United States in 1933 at Canonsburg, Pennsylvania, where uranium was recovered from residues of an earlier radium mill (Markos and Bush, 1981). Increasing military and civilian demands for uranium in the 1940's and 1950's prompted rapid expansion of the uranium milling industry, which localized within uranium districts in the western United States. Cessation of contracts and incentives from the federal government, depletion of higher grade ores, and slow growth of the nuclear utility industry decreased the commercial demand for uranium in the mid- to late-1960's.

This decrease prompted the closure of early uranium mills, many of which were abandoned without reclamation because of the absence of binding regulations for human and environmental protection. In the Uranium Mill Tailings Remedial Action Project (UMTRAP), the U.S. Department of Energy is selecting and performing remedial actions to reduce human and environmental hazards at 24 of these abandoned mills.

Following the uranium milling depression during the 1960's, the demand for uranium increased in the 1970's as nuclear power generation increased. As shown in Figure 2, uranium recovery continues at conventional mills in the states of Colorado, New Mexico, Texas, Utah, Washington, and Wyoming.

Expressing its findings that uranium mill tailings may pose a present and potential threat to humans and the environment, the U.S. Congress passed the Uranium Mill Tailings Radiation Control Act of 1978 (UMTRCA), which provides federal and State authority for regulation of uranium mill tailings. In addition to establishing the UMTRA Project for remedial actions at inactive uranium recovery sites, the Act authorizes the U.S. Environmental Protection Agency (EPA) and the U.S. Nuclear Regulatory Commission (NRC) to promulgate standards for the management of uranium mill tailings.

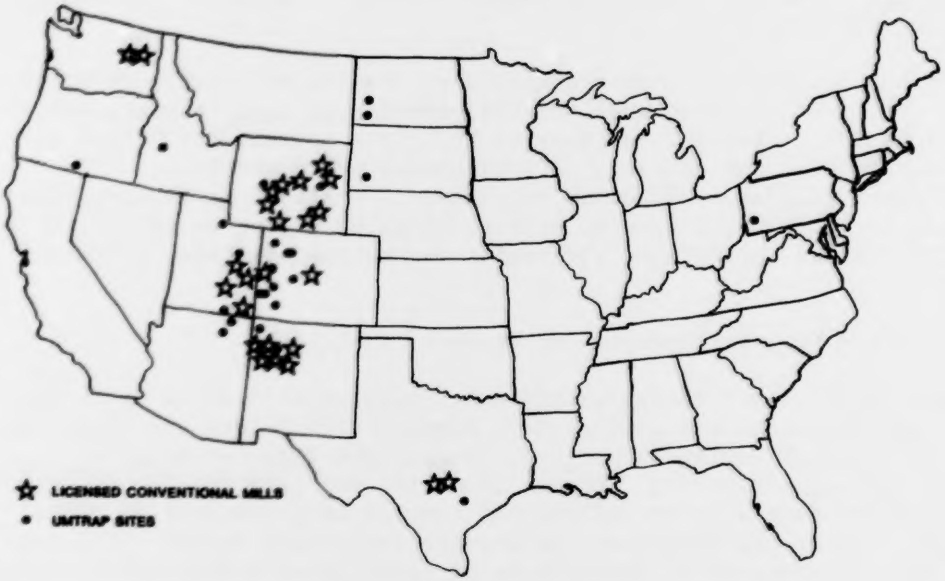

Figure 2. Conventional uranium mill locations in the United States. Licensed mills are represented by stars (White, 1984); abandoned uranium mill sites (UMTRAP sites) are represented by dots (Public Law 95-604).

Disposal of uranium mill tailings and tailings solutions has changed since the early 1950's in response to increasingly stringent regulatory directives. Early uranium mills discharged tailings into impoundments, dry stream valleys, marshes, streams, and rivers. Direct discharge of tailings to surface water continued until as recently as 1977 (White, 1984).

Beginning in the early 1960's, mill operators were encouraged to discharge tailings into impoundments rather than surface water bodies to avoid surface water degradation. Although early investigators recognized that impounding the tailings solutions might affect ground water resources, seepage into the ground was considered an acceptable method for solution disposal. Investigators speculated that ion exchange, adsorption, and chemical precipitation would remove most of the contaminants in the tailings solutions, thus protecting ground water quality. Allowing for seepage from the tailings impoundments also considerably reduced (up to 10 times) the areal extent of impoundments compared with the extent necessary to evaporate all tailings solutions at large mills (Merritt, 1971).

In 1980 the NRC promulgated standards for the management and disposal of uranium mill tailings (e.g., 10 CFR Part 40, Appendix A) under authority granted in UMTRCA. These standards reflected growing environmental concern that disposal of uranium mill tailings in unlined, above-grade surface impoundments generally could not provide the same level of environmental protection provided by lined, sub-grade disposal impoundments. Under these standards, mill operators are required to control the effects of tailings solution seepage on ground water quality through design and corrective actions. In contrast to early tailings impoundments, recent impoundments have been designed and constructed to reduce seepage of uranium mill tailings solutions into aquifers.

Regulations for ground water protection at uranium mill tailings sites became more stringent in 1983, when the EPA promulgated environmental standards that include substantial portions of EPA's regulations for RCRA (Resource Conservation and Recovery Act) hazardous waste facilities. In response to these standards and other regulatory directives, mill operators are assessing the effects of uranium mill tailings on ground water quality to develop effective and appropriate strategies for mitigating these effects.

Effects of Uranium Mill Tailings on Ground Water Quality

The importance of monitoring ground water quality at uranium mills has been recognized since the mid-1960's, when seepage of tailings solutions was considered an acceptable disposal method. During the last two decades, ground water quality monitoring programs at active and inactive uranium mills have characterized partially the effects of uranium mill tailings on ground water quality. These characterizations provide the second portion of a two-part historical perspective of the effects of tailings on ground water quality. A classification of ground water contaminants is introduced below to summarize these effects.

Although the classification encompasses many of the primary constituents of concern that have affected ground water quality at uranium mills, current ground water quality data are insufficient to assess the behavior of organic contaminants that may migrate away from tailings impoundments. Merritt (1971) stressed the potential signficance of these organic compounds in determining environmental impacts of mill tailings seepage.

Inorganic contaminants associated with uranium mill tailings may be categorized into three classes based on the following criteria: contaminant concentrations in mill tailings, their mobilities in hydrogeologic systems, and their observed migration in hydrogeologic systems. This classification summarizes the effects of uranium mill tailings on ground water quality and supports development of ground water monitoring and corrective action programs.

Class I Constituents

Class I constituents include contaminants that are relatively abundant in uranium mill tailings, are mobile in most hydrogeologic systems, and migrate at or about the same velocity as ground water flow. These constituents include sulfate, nitrate, chloride, and calcium. As shown in Table I, concentrations of sulfate and chloride in mill tailings solutions generally exceed 100 mg/l and may exist at concentrations in excess of 5000 mg/l. Concentrations of these constituents in uranium mill tailings solutions may vary because of differences in milling processes and ores (Dreesen, et al., 1983). For example, the concentration of nitrate and/or ammonium would be expected to be significant in mill tailings from uranium mills that use ammonium nitrate to precipitate uranium and to control solution pH. Site-specific evaluations of ground water quality should consider potential variations in the concentrations of Class I constituents caused by variations in milling processes and ores.

Class I constituents are mobile in ground water because they are not attenuated significantly by solid-solute interactions such as adsorption or ion exchange. The ability of Class I constituents to migrate at or about the velocity of ground water flow has been recognized for many years (Freeze and Cherry, 1979). Class I constituents are often included in ground water monitoring programs as indicators of ground water contamination because they migrate in advance of more-attenuated constituents. Although they generally are considered to be innocuous to humans at low concentrations, significant concentrations of Class I constituents may preclude sensitive uses of ground water without proper treatment.

The migration of Class I constituents in ground water affected by uranium mill tailings solutions has been observed at uranium mills (Morin, et al., 1982; Williams and Osiensky, 1983; Cherry, Gillham, and Barker, 1984; Longmire, 1984). The migration of sulfate in ground water at the site illustrated in Figure 3 is considered to be representative of the migration of Class I constituents in shallow ground water affected by uranium mill tailings. Concentrations of Class I constituents have increased several orders of magnitude above background levels in ground water that has been severely affected by seepage of tailings solutions. Because sulfate generally exists at higher concentrations in mill tailings solutions than other Class I constituents, it is preferred as an indicator of ground water contamination from uranium mill tailings solutions.

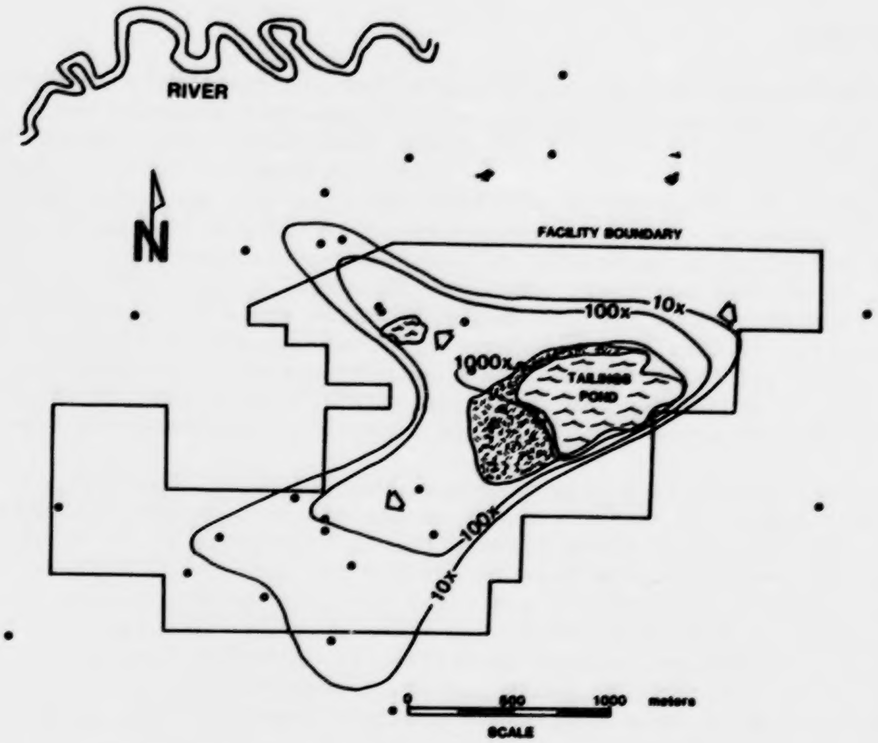

Figure 3. Concentrations of sulfate in shallow ground water affected by uranium mill tailings (shaded) in October and November, 1983. Concentrations are expressed as multiples of an assumed background level of 25 mg/l. Ground water flow directions are indicated by arrows; dots represent monitoring locations (WNI, 1984).

Class II Constituents

Class II constituents are relatively abundant in uranium mill tailings and have variable mobilities and migration potentials in ground water. This class consists of uranium, selenium, molybdenum, arsenic, and vanadium. Concentrations of Class II constituents in uranium mill tailings solutions may exceed 100 mg/l, but they usually are less than 50 mg/l. Similar to the variation in concentrations of Class I constituents, concentrations of Class II constituents may vary because of differences in milling processes and ores. Molybdenum and selenium, for example, often are more concentrated in alkaline-leach tailings solutions than in acid-leach solutions.

In contrast to the behavior of Class I constituents, the migration of Class II constituents in ground water may be attenuated significantly by solid-solute interactions. Migration of these consituents in hydrogeologic systems is controlled partially by speciation of the constituents and their interactions with solids through which they migrate. Speciation of Class II constituents depends on the composition of the ground water, oxidation potential, complex formation (e.g., carbonate, chloride, and sulfate complexes), and kinetics of species transformation. Class II constituents may be mobile in ground water as oxyanions or anionic complexes (Dreesen, et al., 1982). Solid-solute interactions that attenuate the migration of these constituents include precipitation, sorption, and oxidation-reduction reactions. Class II constituents may be considered as the most significant constituents in terms of their effects on ground water quality because of their relative abundance in uranium mill tailings, their toxic characteristics, and their mobilities.

Uranium is present in uranium mill tailings solutions at concentrations between 10 and 100 mg/l. Most of this uranium is soluble in the form of various complexes of uranyl ions (UO_2^{+2}) with sulfates, phosphates, and chlorides (Langmuir, 1981). In ground water affected by mill tailings seepage, uranyl ions may be complexed with carbonate and bicarbonate ions that may be the dominant anions in alkaline ground water common in western uranium districts. Although uranyl ion migration may be attenuated by solid-solute interactions, such as precipitation and sorption, anionic complexing of uranyl ions greatly increases uranium mobility in ground water.

As shown in Figure 4, uranium has migrated significant distances in ground water from tailings impoundments and exists at concentrations significantly above background levels. Similarity of the shapes and relative concentrations of the uranium plume in Figure 4 and the sulfate plume in Figure 3 suggests that sulfate complexing of uranyl ions has contributed to uranium migration from the tailings area. Uranium may exist at concentrations that are several orders of magnitude greater than background levels in ground water affected by uranium mill tailings.

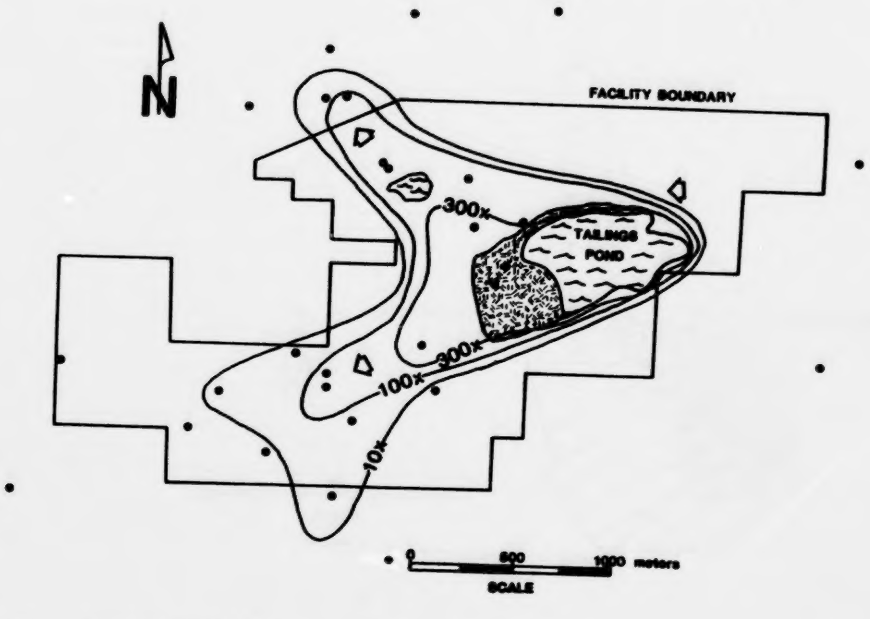

Figure 4. Concentrations of uranium in shallow ground water affected by uranium mill tailings (shaded) in October and November, 1983. Concentrations are expressed as multiples of an assumed background level of 0.03 mg/l. Ground water flow directions are indicated by arrows; dots represent monitoring locations (WNI, 1984).

10 Selenium, a relatively abundant element in uranium mill tailings and leachate, is also mobile in groundwater as several different oxyanions. As shown in Table I, the concentration of selenium in tailings solutions may exceed 30 mg/l, three thousand times its maximum contaminant level in public drinking water (40 CFR Part 141). Selenium may be extremely mobile in neutral and alkaline ground water especially at pH's greater than 7.5 (Peterson, et al., 1983). Selenium speciation controls its mobility and solid-solute interactions that attenuate its migration in hydrogeologic systems. These interactions include precipitation, coprecipitation, and sorption. Serne, Peterson, and Gee (1983) demonstrated the mobility of selenium from uranium mill tailings solutions in laboratory simulations.

Consistent with laboratory simulations, selenium has migrated in ground water significant distances from uranium mill tailings disposal impoundments. As shown in Figure 5, selenium migration has increased concentrations several times above background levels at distances greater than 500 meters from the tailings disposal area. Selenium migration in ground water at this site may have been promoted by the alkaline-leach process used in the mill and low capacities of affected hydrogeologic media to attenuate the migration.

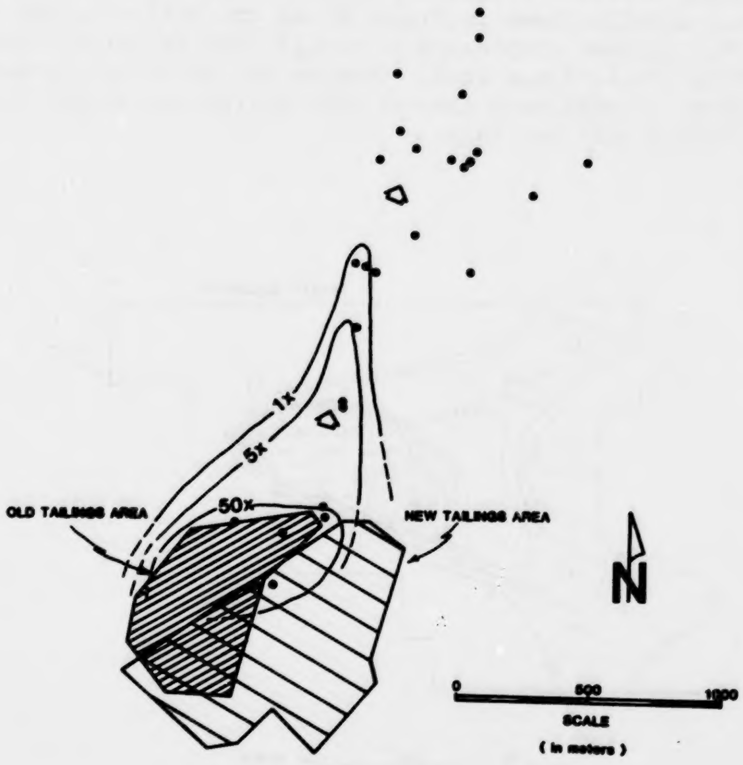

Figure 5. Concentrations of selenium in shallow ground water affected by uranium mill tailings in February, 1980. Concentrations are expressed as multiples of an assumed background level of 0.01 mg/l. Ground water flow is indicated by arrows; dots represent monitoring locations (Wilder, et al., 1983).

Similar to the migration behavior of selenium, molybdenum may be extremely mobile in alkaline ground water. Although molybdenum usually exists at concentrations between 1 and 10 mg/l in mill tailings solutions, its concentrations may exceed 100 mg/l (Longmire, 1983; Runnells, et al., 1983). Molybdenum speciation controls partially its mobility and migration attenuation by hydrogeologic media. Molybdenum typically migrates in shallow ground water as anionic molybdate complexes, which may be attenuated by precipitation or sorption. These attenuative interactions decrease with increasing pH until virtually no sorption occurs above a pH of 7.5 (Peterson, et al., 1983).

As shown in Figure 6, molybdenum concentrations have increased up to several hundred times background levels in ground water affected by uranium mill tailings. Significant increases in molybdenum concentrations were detected in wells that are 1800 meters from the old tailings area within ten years of initial mill operation, thus indicating the high mobility of molybdenum at this site (Wilder, et al., 1983). Runnells, et al., (1983) concluded that selenium was more mobile than molybdenum in ground water at this site because the mobility of molybdenum was controlled partially by ground water pH, whereas selenium was mobile at all simulated pH's. Significant molybdenum concentrations have also been observed in ground water affected by uranium tailings at UMTRAP sites (DOE, 1984).

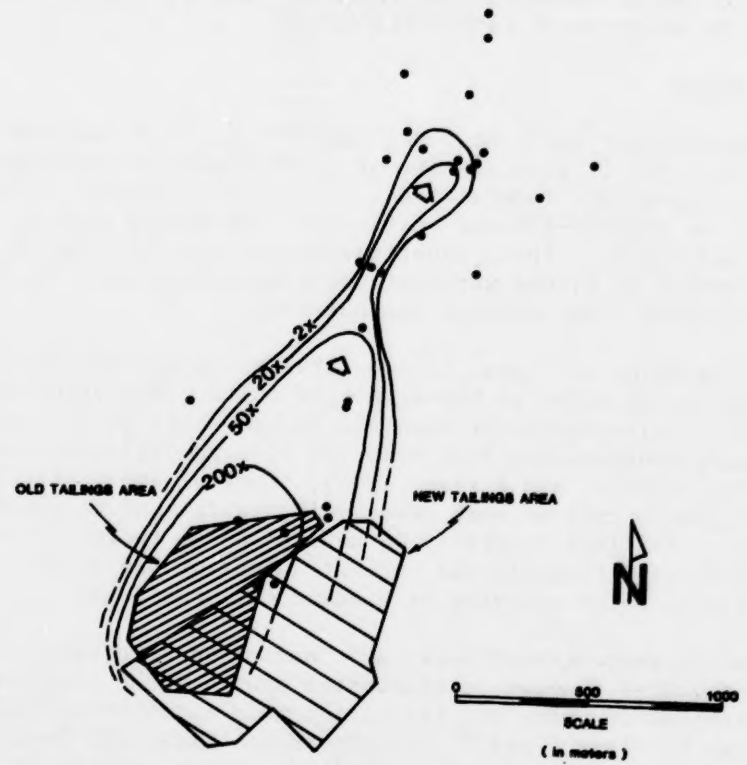

Figure 6. Concentrations of molybdenum in shallow ground water affected by uranium mill tailings in February, 1980. Concentrations are expressed as multiples of an assumed background level of 0.5 mg/l. Ground water flow directions are indicated by arrows; dots represent monitoring locations (Wilder, et al., 1983).

12 In contrast to the behavior of the other Class II constituents, arsenic and vanadium seldom exist at significant concentrations in ground water at large distances from uranium mill tailings impoundments. Arsenic is generally present in mill tailings solutions at concentrations between 1 and 10 mg/l. Although vanadium concentrations in uranium ore may be high enough to allow economical byproduction of vanadium in conventional uranium mill circuits, vanadium concentrations in mill tailings solutions seldom exceed 5 mg/l.

The mobility of arsenic in ground water depends on the pH of the ground water. At intermediate pH's between 3 and 7, arsenic generally migrates as the species $H_2AsO_4^-$, which is the most likely arsenic species adsorbed by clay minerals (Peterson, et al., 1983). In addition to sorption, arsenic migration in ground water may be attenuated effectively by precipitation and coprecipitation. In alkaline ground water arsenic migration may be significant, especially in ground water flowing through hydrogeologic units which contain little or no ferric oxides.

In ground water affected by uranium mill tailings solutions, vanadium generally exists in the +4 and +5 oxidation states with the pentavalent form dominant in ground water at neutral pH's (Peterson, et al., 1983). In more acidic ground water, the quadravalent form of vanadium is more important in determining vanadium speciation and mobility. Similar to the migration behavior of other Class II constituents, vanadium migration in ground water may be attenuated by sorption of ferric hydroxides.

Class III Constituents

Class III constituents exist at lower concentrations in uranium mill tailings than Class I and II constituents or their migration is attenuated significantly by solid-solute interactions. These constituents include radionuclides such as thorium-230 and radium-226, and metals such as lead, copper, cadmium, and nickel. These constituents are not observed at elevated concentrations in ground water affected by uranium mill tailings at significant distances from tailings impoundments.

The limited migration of Class III constituents in ground water may be caused by pH-buffering of affected hydrogeologic units. Buffering of acidic tailings solutions by dissolution of minerals in the units (e.g., calcite and dolomite) may remove contaminants from solution through precipitation and adsorption (Cherry, Gillham, and Barker, 1984). Many of the Class III constituents will precipitate or sorb readily in neutral and alkaline ground water. Buffering of tailings solutions to neutral pH's, therefore, may be expected to decrease significantly the concentrations of Class III constituents in ground water affected by uranium mill tailings.

At uranium mills where ground water pH's have been decreased significantly by tailings seepage, contaminated ground water contains elevated concentrations of heavy and transition metals, radionuclides, and high concentrations of Class I and II constituents. Class III constituents are most concentrated in the cores of contaminant plumes associated with acid-leach tailings, in which ground water pH's generally are less than 4.5. The cores of these plumes advance as the solution exceeds the pH-buffering capacity of affected hydrogeologic media (Markos and Bush, 1981; Morin, et al., 1982; Cherry, Gillham, and Barker, 1984).

Figure 7 shows the decrease of ground water pH near mill tailings at the site discussed previously for uranium and sulfate migration. Comparison 13
of Figures 3, 4, and 7 shows that although sulfate and uranium have migrated significantly beyond 1000 meters from the tailings area, the buffering front and the acidic ground water it contains have advanced less than 500 meters from the tailings. Migration of Class III constituents within the acidic core would be retarded compared to migration of Class I and II constituents in advance of the buffering front. At most uranium mill tailings sites, the zone of significant degradation of groundwater pH is limited because of large pH-buffering capacities of affected hydrogeologic media. Concentrations of Class III constituents generally have not increased above background levels outside of the low-pH cores of contaminant plumes associated with tailings.

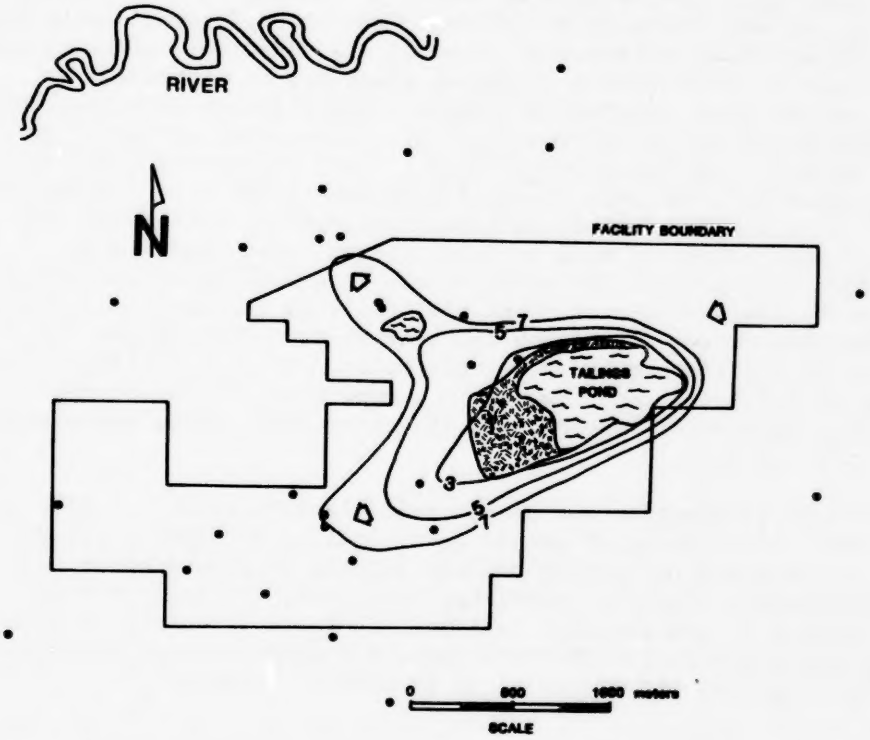

Figure 7. Distribution of laboratory pH values of shallow ground water affected by uranium mill tailings (shaded) in October and November, 1983. Ground water flow directions are indicated by arrows; dots represent monitoring locations (WNI, 1984).

Summary and Conclusions

Disposal of uranium mill tailings in unlined surface impoundments has affected the quality of shallow ground water at conventional uranium mills in the United States. A historical perspective of the effects of uranium mill tailings on ground water quality consists of a review of the history of mill tailings management and an evaluation of the effects of this management on ground water quality.

Uranium mills in the 1950's, 60's, and 70's were discouraged from discharging mill tailings solutions into surface water to reduce water quality degradation and obvious environmental impacts. Increasingly stringent environmental regulations in the 1980's have prompted mill operators to protect ground water quality through impoundment design and programs to correct existing ground water quality degradation.

The effects of uranium mill tailings on ground water quality may be summarized using a contaminant classification based on the abundance of the contaminants in the tailings, their mobilities in ground water, and their observed migration in affected hydrogeologic systems. Class I constituents include sulfate, chloride, nitrate, and calcium, which are abundant in tailings, mobile in ground water, and have been observed to migrate significant distances at significant concentrations away from tailings impoundments. Class II constituents are less abundant in tailings than Class I constituents and include uranium, selenium, molybdenum, arsenic, and vanadium. Although the migration of Class II constituents in ground water may be attenuated by solid-solute interactions, uranium, molybdenum, and selenium have been observed to migrate significant distances from tailings impoundments at concentrations significantly greater than background levels. Class III constituents are less abundant in uranium mill tailings compared with constituents of Classes I and II. Class III constituents include radionuclides such as radium-226 and thorium-230, as well as metals such as lead, copper, zinc, and nickel. Significant migration of Class III constituents in ground water appears to be confined within contaminant plumes where mill tailings seepage has decreased groundwater pH. The extent of this pH decrease is dependent on the pH buffering capacity of affected hydrogeologic media. At many tailings sites, concentrations of Class III constituents in ground water do not exceed background levels outside of low-pH cores of contaminant plumes associated with uranium mill tailings.

The historical perspective and contaminant classification provided in this paper support development of ground water quality monitoring programs and selection of programs to correct adverse effects on ground water quality. Historical perspectives about the mobility, transport, and fate of toxic constituents present in uranium mill tailings and their effects on ground water quality are essential to effective waste management as environmental strategies shift emphasis from remedial to protective actions.

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Biographical Sketches

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William Dam (U. S. Nuclear Regulatory Commission, Mail Stop 623-SS, Washington, DC, 20555) joined the Division of Waste Management after obtaining a Bachelor's degree in Geology from Guilford College and a Master's degree in Geology from the University of Wyoming. As a project manager/ geochemist at NRC, Bill's interests include contaminant migration, geochemical modeling, hydrogeology, and field investigations.

BACKFILLING TECHNIQUES AND ALKALINE ADDITION TO CONTROL ACID MINE DRAINAGE IN A COAL STRIP MINE

Neil W. Hedrick, Edgar W. Meiser, Jr., and Robert M. Hershey

Antrim Mining, Inc.
Blossburg, Pennsylvania
Meiser & Earl / hydrogeologists
State College, Pennsylvania

Antrim Mining, Inc. in Tioga County, Pennsylvania was faced with developing techniques to control acid mine drainage (AMD) in a proposed strip mine adjacent to a deep mine complex. As much as 40 feet (12.2m) of the total overburden contained shale high in pyrite; this shale is a known AMD-producer.

Four separate problem areas were identified. To prevent oxidation of the pyrite, and allow the discharge from the strip mine to meet state regulations, a waste product from lime manufacturing was used in an attempt to maintain alkaline conditions. A sandstone underdrain was constructed to minimize saturated ground water conditions and to direct ground water either to monitoring sumps before release into the abandoned deep mines or toward a common discharge point for treatment. The underdrain capacity was designed to exceed the recharge rate by more than an order of magnitude. Runoff will be promoted and recharge thus reduced by grading the backfill into rolling swales with drainageways floored on compacted glacial till.

The fourth task was to develop a materials-handling plan which allowed for the segregation of acidic spoil and selective placement of the alkaline material. The plan developed was economically feasible, under the coal company's contract price commitment, and practical in terms of the available mining equipment and stratigraphic sequence of the mine site.

Initial water quality monitoring data from the active unreclaimed mining operation and the performance of the rock underdrain are encouraging. The final evaluation of this experimental mining plan cannot be made until backfilling, final grading and revegetation are completed in the next two years.

BACKGROUND

Antrim Mining, Inc. of Blossburg, Pennsylvania has developed an 85-acre (34.4ha) bituminous coal strip mine site near the town of Antrim in

Tioga County. Because of the serious acid mine drainage (AMD) problems associated with strip mining in this geologic setting, and because the site lies adjacent to a large deep mine complex, innovative techniques were required to control AMD in order to obtain a mining permit from PA Department of Environmental Resources (DER).

Geology

The Allegheny Group is the major coal-bearing stratigraphic unit of the north-central coal fields of Pennsylvania. In ascending order, the local coals being mined at this site are equivalent to the Lower Kittanning or B-coal and the Middle Kittanning or C-coal. More than 28 drill holes were logged at the mine site, so the stratigraphy is well-defined; Figure 1 is a generalized geologic section of the site. Drilling with a reverse-circulation, air rotary drill rig allowed accurate collection of discrete stratigraphic intervals and drilling below the deep mine in the upper split of the B or Bloss coal.

We sampled and analyzed three overburden holes according to the procedures outlined by Sobek, et al. (1978). The rock chips were analyzed for total and pyritic sulfur percentage. Since past experience suggested there was no significant amount of alkaline material, and because no dependence was placed on any naturally occurring alkalinity to control the pH of water in the mine backfill, neutralization potential of the overburden was not analyzed.

The stratigraphic interval from the upper Bloss rider coal down to the Bear Creek coal consistently contains numerous zones of high pyritic sulfur, ranging up to 9.25% and averaging about 2.5%. Above this zone the pyritic sulfur is consistently low, ranging from 0.03% to 0.23%.

Glacial drift covers the surface of the entire mine site and varies from 2 to 30 feet (0.6-9m) in depth. Sieve analyses performed on four samples show that the glacial drift is a gravelly loam to a gravelly sandy loam. Overburden analyses indicate that total sulfur content is less than 0.08%.

Structurally, the site is located about 2600 feet (793m) south of the Blossburg synclinal axis which has a strike of N60°-80°E. Figure 2 shows the location of the site in relation to the structure contours on the Bloss coal as shown by Kantz (1976). Within the strip mine site dip ranges from north to northwest averaging 5-6%, with bedrock strike paralleling the synclinal axis. As much as 8 feet (2.4m) of structural relief was observed along strike in local "rolls" on the Bear Creek coal. As the coal approaches its structural outcrop to the south, dips steepen up to 11%.

Mining History

Deep mining in the site area began more than 100 years ago and continued into the early part of this century. Approximately 1360 acres (550ha) of Bloss coal was mined in the Antrim No. 1 deep mine, which lies immediately adjacent to the northern, downdip boundary of the strip mine site (Figure 2).

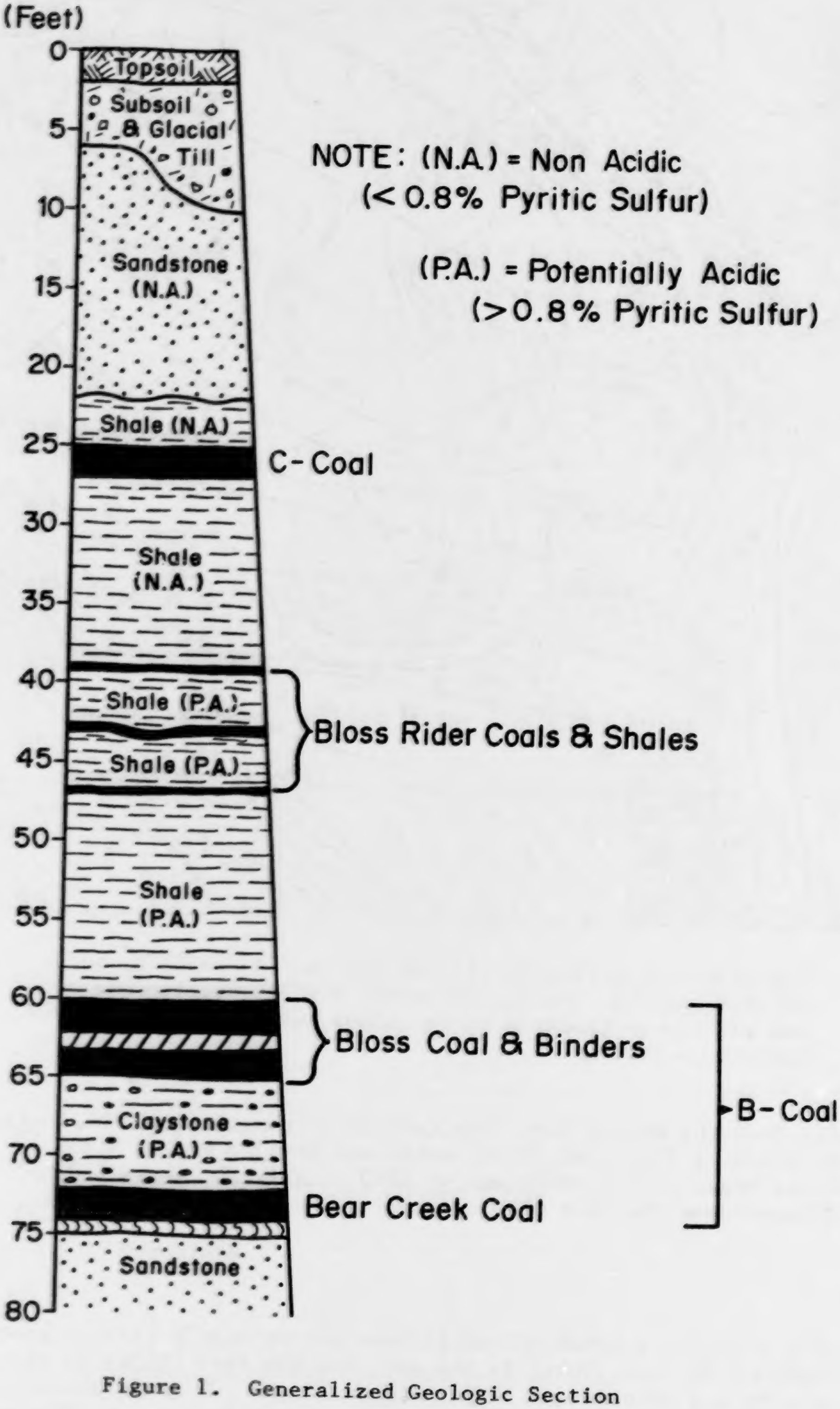
(Feet)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
Topsoil
Subsoil & Glacial Till
Sandstone (N.A.)
Shale (N.A.)
C-Coal
Shale (N.A.)
Shale (P.A.)
Shale (P.A.)
Bloss Rider Coals & Shales
Shale (P.A.)
Bloss Coal & Binders
B-Coal
Claystone (P.A.)
Bear Creek Coal
Sandstone
NOTE: (N.A.) = Non Acidic
(< 0.8% Pyritic Sulfur)
(P.A.) = Potentially Acidic
(> 0.8% Pyritic Sulfur)

Figure 1. Generalized Geologic Section

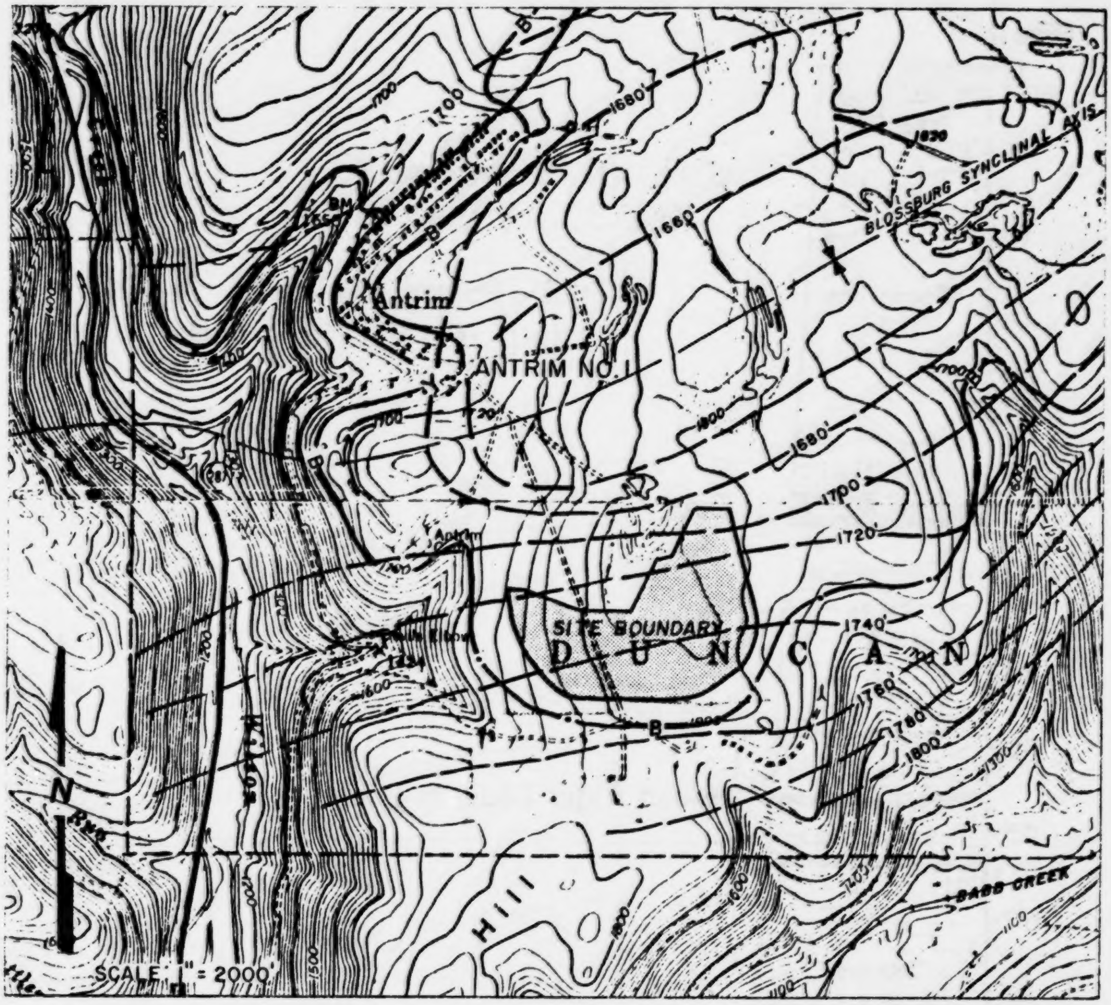
BLOSSBURG SYNCLINAL AXIS
1680'
1660'
1680'
1700'
1720'
1740'
1760
1780
1800
Antrim
ANTRIM NO.1
SITE BOUNDARY
D U N C A N
BABB CREEK
HILL
N
B
SCALE 1" = 2000'

Figure 2. Topographic map from Morris and Antrim, PA USGS 7½' quadrangles showing location of Antrim Mining's strip mine site and structural contours on the Bloss coal from Kantz (1976). Scale 1" = 2000'.

Ground water from the entire deep mine complex discharges from the Antrim No. 1 drift which is 3000 feet (915m) north and downdip of the site. Surface mining began in the 1940s and by 1970 roughly 500 acres (202ha) had been disturbed on the four mineable coal seams in the deep mine area.

Hydrology

The site occupies a broad upland plateau having nearly flat to gentle slopes. Within 1000 feet (305m) to the east and 600 feet (183m) to the west of the site are precipitous slopes approaching a 50% grade and dropping some 500 feet (152m) in elevation to Wilson Creek and Babb Creek (Figure 2).

There are no defined perennial surface streams draining this upland. Ephemeral springs flow on the southern and western sides of the mine area, fed by the shallow glacial till aquifer.

Detailed stream flow and mine discharge records kept by the U.S.G.S. from 1978 through 1982 on a nearby upland glaciated watershed, underlain by extensive deep mining on the Bloss coal, provided an analogy for the Antrim strip mine site. One deep mine discharge in this analogous setting, the Hunter Drift, drains some 400 acres (162ha) of Bloss coal deep mine, overlain by large areas of unreclaimed open strip mining. Discharges during water years 1978, 1979 and 1980 from Hunter Drift ranged from 20.4 to 29.5 inches (51.8cm to 74.93cm) or 47% to 74% of total annual precipitation was recharged to the deep mine system. Based on Penn State University research in the Bloss Coal Basin currently underway by D.R. Buss (1982 personal comm.), recharge in a 1940's poorly-reclaimed strip mine in his research area was about 25% of precipitation or about 10 inches (25.4cm) per year.

In order to define the ground water flow systems of this site, we installed nested piezometers in four test holes. Our interpretation of the water levels in the piezometers suggests that there are three major ground water flow systems in the site area. The upper system is found in the various perched stratified zones within the glacial till. The degree of interconnection with lower systems and extent of true perching of the shallow till ground water is seasonally variable.

The intermediate bedrock system above the Bloss coal shows the underdrain effects of the nearby Bloss deep mine. There is a steep hydraulic gradient to the north, down structural dip towards the deep mine.

The lower ground water system is found in the thick sandstones below the Bear Creek coal where flow follows the regional structure. Recharge to this zone is primarily from leakage through the overlying Bloss and Bear Creek underclays.

Ground water discharge from this site occurs in three primary areas. The ground water system in the till contributes to ephemeral springs. Ground water in the intermediate system above the Bear Creek coal eventually drains downdip to the open Bloss deep mine workings and discharges from the main heading of the Antrim No. 1 deep mine. Subsidence fracturing in the strata overlying the deep mine enhances vertical drainage to this vast underdrain system. The lower sandstone system below the B-coal drains primarily downdip and contributes to the base flow of the stream in the incised valley near the axis of the syncline. This stream also receives the discharge from the Antrim No. 1 deep mine.

The water quality of discharges immediately around the site area that have not been affected by previous mining are generally good, with specific conductances of less than 60 umhos/cm, sulfates less than 20 mg/l, acidity less than 50 mg/l, and iron generally below 0.3 mg/l. The Antrim No. 1 discharge is the single largest ground water discharge in the area. This and other deep mine discharges west of Antrim render Wilson Creek unsuitable for any fish life (Figure 2). Table 1 shows the range of flows and water quality analyses of the Antrim No. 1 deep mine discharge. Flows are measured with a calibrated V-notch weir each time a water quality sample is collected. Monthly samples were collected from February 1981 to October 1982; since November 1982 weekly samples have been collected.

Table 1. Flow and Water Quality from Antrim No. 1 Deep Mine Discharge.

	Low	High	Mean
Flow gpm (l/sec)	326 (20)	5030 (317)	1130 (71)
pH units	2.9	3.5	3.1
Acidity mg/l	140	380	230
Iron mg/l	1.6	34	12
Manganese mg/l	2.9	36	5.2
Aluminum mg/l	1.8	180	12
Sulfate mg/l	130	520	270
Specific Conductance umhos/cm	380	880	680

TECHNIQUES TO CONTROL AMD

The design team for the strip mine permit application took a unique point of view toward a coal mine in that the first priority was to design a backfill plan. We decided that a redundant system of AMD prevention measures would be used since no single measure could be counted on to completely eliminate AMD as mandated by Federal and State law. The basic measures to be employed were:

1. chemical control of pH and neutralization of AMD in the backfill by alkaline addition (lime blending)
2. hydrologic control of ground water in the backfill to minimize saturated conditions and route ground water flow beneath the acid-forming overburden
3. runoff control to minimize infiltration to the backfill
4. a materials handling plan to allow segregation of acidic spoil and selective placement of alkaline material that is economically feasible and practical.

Chemical Theory of Lime Addition

Acid mine drainage has been and continues to be a major problem generated by the mining of coal. Although some of the basic chemistry is understood as a result of research over the past twenty years, Rose, et al. (1983) concluded that the process is complex enough that we are not yet able to make accurate predictions of future acid generation at mine sites or to prevent or ameliorate acid drainage at an economically acceptable cost.

This project attempts to economically control the generation of AMD by maintaining a pH above 7 in the backfill by the addition of a waste lime material. Microorganisms control the overall rate of the AMD reaction by "mediating the oxidation of ferrous iron since it, alone, is the rate determining step" (Stumm & Morgan, 1970, p. 542). Controlling the growth of the microbes *Thiobacillus* and *Ferrobacillus ferrooxidans* can greatly reduce AMD production, possibly by one million times; at a pH of greater than 5.5, the catalytic effect of the bacteria becomes insignificant (Singer and Stumm, 1969). As long as the water infiltrating through the mine spoil backfill is maintained at a pH above 7.0, the AMD reaction is suppressed since the pyrite, no matter what its concentration, is nearly unavailable for reaction.

According to the work done by Rose, et al. (1983), acidity produced from the AMD reaction is strongly related to the pyritic sulfur content of the spoil material. Overburden containing less than about 1% pyritic sulfur generates "negligible" amounts of acidity (p. 39). A more conservative approach was used in this project in that any overburden material containing more than 0.8% pyritic sulfur was considered potentially acid-producing. Figure 1 shows the acid and non-acid-producing zones in the overburden. Leaching tests done by Rose, et al. (1983) show that calcite as a layer above the acid-producing rock produces a greater reduction in both acidity and sulfate than leaching tests where the calcite was mixed with the overburden sample. This suggests that the relatively alkaline water produced by passing through the calcite inhibits the acid-producing reaction, rather than just neutralizing the acid after it is produced.

At this site, lime addition is incorporated both above the acid-producing spoil and within this zone in the backfill. The waste lime material used in the alkaline addition to the Antrim backfill is from the baghouse of a large lime manufacturing plant. The waste is 20% CaO, 57% $CaCO_3$ and 23% inert. Since the waste lime is stockpiled outside, most of the CaO has slaked to $Ca(OH)_2$ (hydrated lime); 20 tons (18t) of CaO is equivalent to 26 tons (24t) of $Ca(OH)_2$. The approved mine design required that 400 tons (362t) of waste lime be added to the backfill material for every acre (0.4ha) mined. An additional 50 tons/acre (112t/ha) was spread on the Bear Creek coal pavement.

The dissolution of the waste lime, which will maintain a pH greater than 7.0, will determine the longevity of the alkaline material in the backfill. Work in a similar geologic setting near the study area, described earlier, indicates that 25% of precipitation or about 10 inches (25.4cm) of ground water recharge per year in a poorly-reclaimed strip mine is realistic. As will be discussed later, this is a conservatively high estimate for the Antrim site since techniques are included to maximize runoff at this site.

If all of the 10 inches (25.4cm) per year that becomes recharge reaches an initial alkalinity of 1000 mg/l as $CaCO_3$ as it dissolves the lime, then 1680 pounds (762kg) of $Ca(OH)_2$ will dissolve per year per acre (0.4ha). The equivalent of 104 tons (94t) of $Ca(OH)_2$ is added to the backfill with every 400 tons (362t) of waste lime added. Thus, it should take at least 124 years for the water that infiltrates through the backfill to dissolve all of the hydrated lime. As the lime is depleted, the recharge water will reach a concentration of about 100 mg/l as $CaCO_3$ in contact with the limestone portion of the baghouse lime. Under these conditions, 227 pounds (103kg) of limestone will dissolve per year per acre (0.4ha). Ideally, it would take approximately 2000 years for all of the limestone to dissolve at this rate.

Sandstone Underdrain

A primary objective of the backfill plan is to minimize contact of the acid-producing mine spoils with ground water, based on the assumption that it takes water to make mine drainage. Starting from the bottom up, a 3-foot (1m) thick blanket of coarse sandstone blocks is dumped on the mine floor. This provides a rock blanket underdrain beneath the overlying spoil. The mining plan calls for block cuts oriented parallel to structural dip and for mining updip to produce a "furrowed" effect in the backfill. This, in combination with the rock blanket drain, facilitates drainage on the Bear Creek coal pavement such that no ground water builds up in the spoil.

The post-mining outlet for the downdip drainage is provided by the open, free-draining deep mine system, seen in the northern downdip highwall. During mining and initial backfilling, the pit drainage is collected in sumps along the open highwall cut to hydrologically isolate the active strip mine from the deep mines. In the event that monitoring of this sumped drainage shows water quality unfit for release into the deep mines, it can be routed via rock drains to the northwestern corner of the permit for discharge or treatment.

The hydraulic conductivity of the sandstone rock drain appears to be far more than adequate to handle backfill drainage. Consider downdip drainage along an average mine pavement slope of 5% at a depth of 1 foot (0.3m) saturation in the rock blanket. Textbook hydraulic conductivities for natural sand and gravel deposits range up to 10,000 gallons per day per square foot (gpd/ft^2) (0.47cm/s), while clean gravels exceed 100,000 gpd/ft^2 (4.7cm/s) (Todd, 1959, p. 53). Saturated flow through this material can be described by Darcy's Law:

$$Q = KiA$$

where:
K = hydraulic conductivity (gpd/ft^2) (cm/s)
i = hydraulic gradient (fraction)
A = cross sectional area of flow (ft^2) (m^2)
Q = flow (gpd) (l/s).

As an example of flow for a gradient of unity (100%) through a saturated column 1 square foot in cross-section of sand and gravel with K = 10,000 gpd/ft^2:

$$Q = 10{,}000 \text{ gpd/ft}^2 \times 1 \times 1 \text{ ft}^2$$
$$Q = 10{,}000 \text{ gpd or } 7 \text{ gpm } (0.44 \text{l/s}).$$

At a field gradient of 5%, such as found down the dip of the Antrim strip mine pavement, flow in each 1-foot width of rock drain, saturated to a depth of only 1 foot, and assuming a K = 10,000 gpd/ft^2 (typical for clean sand or sand and gravel) will be:

$$Q = 10{,}000 \text{gpd/ft}^2 \times 0.05 \times 1 \text{ ft}^2$$
$$Q = 500 \text{ gpd or } 0.35 \text{ gpm } (0.022 \text{l/s}).$$

As the width (parallel to strike) of the mining operation is some 2700 feet

(823m), the rock drain capacity at 1-foot saturation depth flowing down an average dip of 5%, still assuming K = 10,000 gpd/ft^2, is:

$$Q = 0.35 \text{ gpm/ft of width} \times 2700 \text{ ft wide} = 950 \text{ gpm } (601/s).$$

Assuming a hydraulic conductivity of 100,000 gpd/ft^2, typical for clean gravels as cited earlier, increases the drain capacity by 10 times to over 9500 gpm (6001/s).

Intuition suggests that tabular blocks of sandstone 1/2 to 1-foot (0.1-0.3m) thick will drain at even higher flow rates than will a sand and gravel deposit. B.K. Hough (1969, p. 76) describes a material called "one-man stone" composed of 4 to 12-inch (10-30cm) diameter stone aggregate having an "effective size" of 6 inches (15cm). From our observations, this is on the average similar to the C-sandstone rubble seen at Antrim's operation. The permeability of "one-man stone" is shown to be 30cm/sec (640,000 gpd/ft^2). Using this value for hydraulic conductivity in our previous calculations yields a rock drain capacity of:

$$Q = KiA$$
$$Q = 640{,}000 \text{ gpd/ft}^2 \times 0.05 \times 2700 \text{ ft}^2$$
$$Q = 60{,}000 \text{ gpm rock drain capacity } (38001/s)$$

where:
A = 2700 ft wide x 1 ft deep
i = 5%

Assuming 10 inches (25.4cm) per year of effective recharge to the backfill as described in earlier sections, this is equal to an average input of about 0.5 gpm/acre (0.0131/s/ha). Distributed over 85 acres (34ha) of strip mine this totals only 43 gpm (2.71/s), far less than the capacity of the drain, which is one to three orders of magnitude greater than the anticipated flow.

Control of Infiltration to the Backfill

By considering the acid-producing mine spoil as a type of waste product, we developed a landfill-type of backfilling program. This includes minimizing infiltration by utilizing a low-permeability cap. This approach seemed appropriate to reduce "leachate" generation, in this case mine drainage. The samples of till tested in the laboratory showed reworked permeabilities at 90% Standard Proctor densities (attainable in the field with conventional equipment) that were about 1×10^{-7}cm/sec. Curiously enough, this is the same value U.S. EPA requires for landfill liners.

A compacted till layer 1 to 2 feet (0.3-0.6m) thick below the final 4 feet (1.2m) of subsoil and topsoil should minimize infiltration recharge to the mine spoil. By grading the backfill into rolling swales with drainageways floored on the compacted till layer, lateral soil drainage will be facilitated above the backfilled mine spoil.

As in the case of landfill management, the cover capping system need not be a perfect liner; "leaks" are acceptable as they represent only a small portion of the total volume of water which is precluded from entering the backfilled mine. 10

Decreasing recharge significantly has the added benefit of reducing the required drainage capacity of the rock underdrain system, and of increasing the longevity of the availability of alkaline materials (lime and limestone) to infiltrating ground waters. The predicted rates of consumption (dissolving) of lime and limestone previously calculated were based on infiltration recharge of 25% of precipitation or 10 inches (25.4cm) per year. If the proposed compacted till cap can eliminate only half of this conservatively high estimate, the "life" of the alkaline materials will double.

Materials Handling Plan

Materials handling is ordinarily the primary concern of the mine designer, but in our case it was secondary to environmental concerns. As previously discussed, we found that the overburden above the C-coal seam was low in pyritic sulfur, hence, low in AMD potential. This meant that it could go anywhere in the backfill. Standard area mining practice would have been to place this low-sulfur overburden in the open strip cut first, thus it would have been on the bottom of the backfill. Our plan, though, called for non-acid-forming overburden to be on the top of the backfill. Figure 3 shows the sequencing of the backfill. We came up with several different methods of accomplishing this sequence using draglines, trucks and loaders for the rock overburden. Scrapers are used to haul glacial till and topsoil around the end of the active cut to the backfill area where they simultaneously spread and compact this material.

The backfilling sequence begins by placement of a lime blanket, at the rate of 50 tons per acre, (20.2ha) on the Bear Creek underclay (Figure 3, Zone G). After blasting the sandstone overlying the C-coal in the adjacent active cut, a dragline casts the sandstone into the empty pit where a dozer spreads it to construct the rock underdrain (Figure 3, Zone F-G). For areas of the mine where the dragline cannot reach, trucks haul the shot sandstone to make the underdrain. The small amount of excess sandstone not used in the underdrain has lime spread on it and is then hauled to the backfill area where it is placed on top of the acid-forming spoil (Figure 3, Zone C-D). Approximately 150 tons (136t) of lime is spread on the glacial till before it is hauled into Zone C-D.

Overburden tests showed that the 35 feet (10.7m) of shale between the C-coal and Bloss coal had several zones containing high pyritic sulfur. These zones were not consistent nor were they readily identifiable in the field. We decided to classify the entire 35 feet (10.7m) between the C-coal and Bloss coal as an acid-producing zone to simplify materials handling.

A dragline positioned on the C-coal pavement in the active cut casts all the zone below it into the adjacent open pit. The dragline also accomplishes lime blending. Lime is spread on the C-coal pavement at the rate of 200-300 tons/acre (448-673t/ha). As the dragline removes the overburden beneath it, the lime is proportionally blended into the overburden, then placed in the zone above the rock drain (Figure 3, Zone E-F).

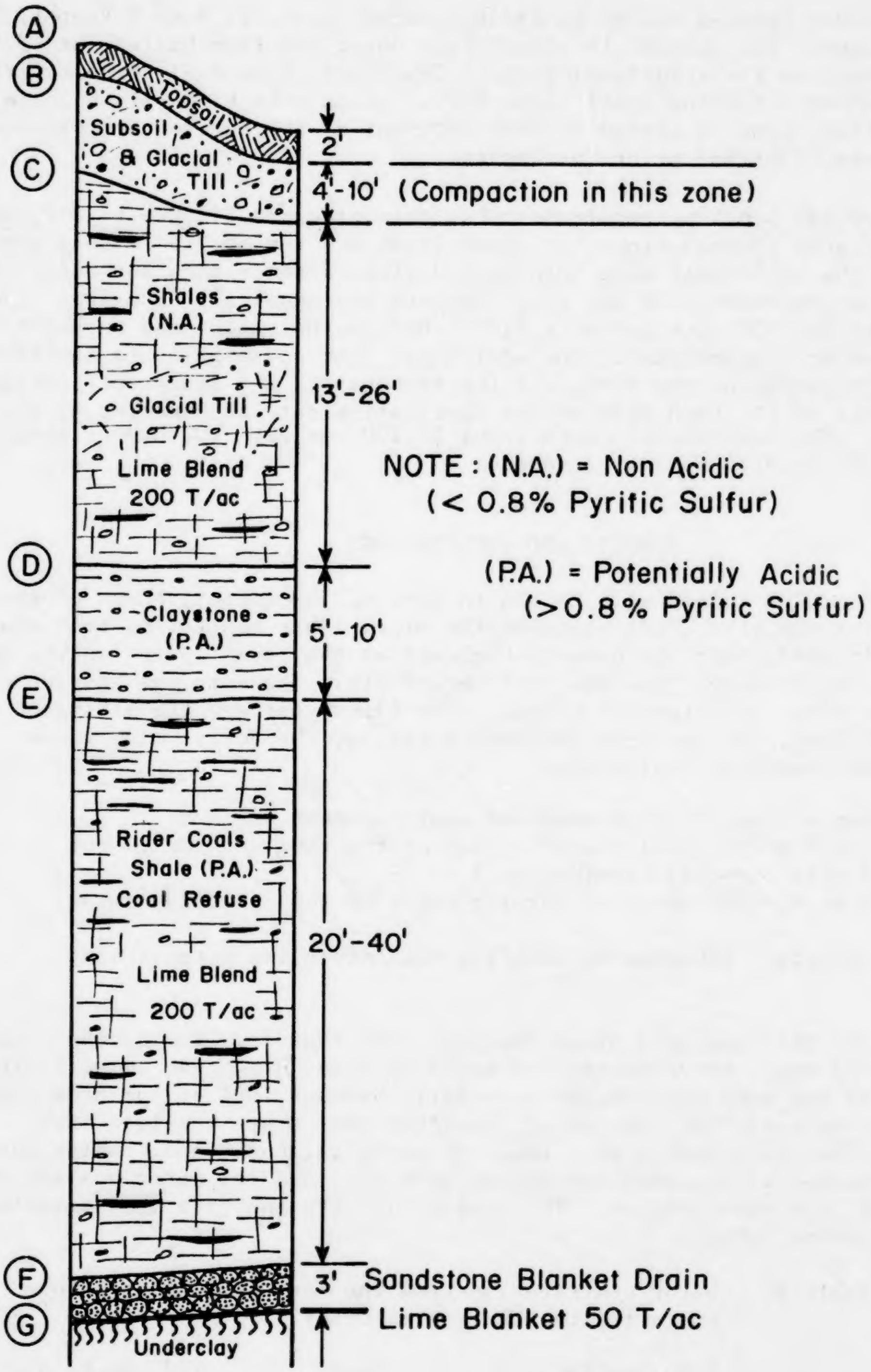
A
B
Topsoil
Subsoil
& Glacial
Till
2'
C
4'-10'
(Compaction in this zone)
Shales
(N.A.)
Glacial Till
Lime Blend
200 T/ac
13'-26'
NOTE : (N.A.) = Non Acidic
(< 0.8% Pyritic Sulfur)
D
(P.A.) = Potentially Acidic
(>0.8% Pyritic Sulfur)
Claystone
(P.A.)
5'-10'
E
Rider Coals
Shale (P.A.)
Coal Refuse
Lime Blend
200 T/ac
20'-40'
F
3'
Sandstone Blanket Drain
G
Lime Blanket 50 T/ac
Underclay

Figure 3. Backfill Sequence

The binder between the Bloss and Bear Creek coals is 5 to 7 feet (1.5 to 2.1m) thick. This binder is ripped by a dozer and then hauled out of the pit and dumped on the acid-forming spoil (Figure 3, Zone D-E) to be covered later by non-acid-forming spoil (Zone C-D). Since this binder is high in pyritic sulfur, lime is spread on this interval at the rate of approximately 150 tons/acre (336t/ha) prior to ripping.

The special handling requirements at this mine add \$10,000 to \$12,000 of cost per acre (0.4ha) mined, or about \$1.00 per ton (0.9t) of coal produced. If the coal seams were thinner and yielded fewer tons per acre, the special handling could make the whole process uneconomical. The lime blend was designed for 450 tons per acre (1000t/ha); actual usage has averaged 650 tons per acre (1460t/ha). The additional lime consumption is caused by difficulty in metering the lime, and the tendency of the equipment operators to always err on the high side of the application rate as directed by mine management. The lime itself costs about \$5,200 per acre (0.4ha) or about \$.50 per ton (0.91t) of coal produced.

RESULTS AND IMPLICATIONS

Antrim Mining agreed with PA DER to monitor the effectiveness of the mine plan for one year after starting the mine. This monitoring took place in a sump located along the downdip highwall at the lowest point on the job which received drainage from the rock underdrain. The sump was developed immediately after the start of mining. The first year saw the mining of 18 acres (7.3ha). At any time during the lift of the active mine there will be four stages of reclamation:

1. open active pit with coal and coal pavement exposed
2. acid-forming spoil placed on top of the rock underdrain but not covered with non-acid-forming spoil
3. rough-graded, non-acid-forming spoil on top of acid-forming spoil
4. completed, revegetated backfill with diversion ditches and topsoil.

Water in the sump is a blend from all four sources. During the early stages of the mine, the completed revegetated area, stage #4, is negligible but grows as the mine matures and eventually becomes 100% of the area when mining is completed. At the end of the first year there was less than 8 acres (3.2ha) in stage #4 of a total 18 acres (7.3ha) mined. Water quality was measured in the sump and in the open pit, and the water quality of the rainfall was also checked. These water quality analyses are summarized in the following tables.

Table 2. Water contribution from the open active pit before contact with alkaline material.

pH	2.8	units	iron	127	mg/l
conductivity	1700	umhos/cm	manganese	18.9	mg/l
alkalinity	0	mg/l	aluminum	43.7	mg/l
acidity	714	mg/l	sulfate	1440	mg/l

Table 3. Range and median values of water quality analyses from 15 samples of the downdip highwall sump, collected during first year of mining.

		Low	High	Median
ph	units	5.1	11	7.4
conductivity	umhos/cm	230	850	710
alkalinity	mg/l ($CaCO_3$)	15	66	68
acidity	mg/l ($CaCO_3$)	0	78	11
iron	mg/l	<0.1	1.8	0.15
manganese	mg/l	<0.1	12.3	8.4
aluminum	mg/l	<1	5.3	<1
sulfate	mg/l	56	530	350

Table 4. Average quality of five rainfall samples.

pH	4.9 units
acidity	45 mg/l
alkalinity	13 mg/l

As can be seen from these data the water quality was altered from the time it entered the mine site as rainfall to its appearance in the sump. The alkaline quality of the sump which includes the severe AMD in the open pit floor (Table 2), implies that most of the drainage from the backfilled area must be strongly alkaline. Water quality standards as set by U.S. EPA for discharges from mining sites are generally met in the sump water for pH (6-9 units), acidity (no net acidity) and iron (3.0 mg/l, 30-day average), but not for manganese (2.0 mg/l, 30-day average). The high levels of manganese in the sump water are coming from the open pit AMD. When the sump pH exceeds 8, then the manganese should precipitate out. As the mine area becomes larger, the open pit acreage will remain basically the same, but the backfill acreage will grow, so that backfill water contribution to the sump will increase in relation to the runoff from the open cut. Hence, the resulting sump water pH and alkalinity should go up. DER allowed the sump area to be backfilled during January 1984, as the unbackfilled pit was detracting from reclamation performance.

Initial water quality monitoring data from the active unreclaimed mining operation and the performance of the rock underdrain are generally encouraging. The final evaluation of this experimental mining plan cannot be made until backfilling, final grading and revegetation are completed in the next two years.

Factors which allowed implementation of this experimental plan to control AMD at the Antrim mining site are:

1. source of low-cost lime
2. favorable stripping ratio of 11.5 to 1 (special handling and lime costs boosted the total mining costs to an apparent ratio of 16 to 1)
3. long-term coal sales contract allowed uninterrupted mining at a fixed price (without this the recent coal market depression would have made mining uneconomical)
4. Abundant glacial till and topsoil to cap the backfill

5. Antrim Mining management commitment to carry out the complex mining plan successfully.

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EVALUATION OF MANAGEMENT PRACTICES FOR MINE SOLID WASTE STORAGE, DISPOSAL, AND TREATMENT

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Abstract

Recent Federal law [RCRA Sections 8002 (f) and (p)] requires the U.S. EPA to study the adverse effects of solid wastes from active and abandoned surface and underground mines on the environment. To support EPA's mandate, PEDCo Environmental, Inc., and its principal subcontractors, Geraghty & Miller, Inc. and Skelly and Loy, Inc., conducted field investigations (EPA Contract No. 68-01-2900) to collect ground water and surface water quality data associated with waste handling practices at eight mines across the United States. Waste management practices for the following waste types were monitored: (1) clay slimes and sand tailings from phosphate mining in Florida, (2) tailings from lead/zinc mining in Missouri, (3) tailings from gold/silver mining in South Dakota and Nevada, (4) mine water from uranium mining in New Mexico, (5) tailings from copper mining in Arizona, (6) copper dump leach material in New Mexico, and (7) overburden from phosphate mining in Idaho.

An extensive industry review including waste sampling and analysis at 65 mines and benefication facilities was undertaken prior to selection of the eight sites for field investigation. The field investigations included installation of ground water monitoring wells and stream sampling stations and collection of samples on five different occasions extending over a period of six to nine months. Collected water quality and hydraulic data were evaluated using statistical and other methods to determine the effects of the waste management practices on the environment.

Seepage of contaminants from waste management practices was identified in ground water at each of the seven sites where ground water was sampled. The principal contaminants identified included sulfate, chloride, and TDS. Heavy metals, radionuclides, or cyanide from waste management practices were detected above background levels in only

isolated cases. Generally the levels of contamination resulting from the waste disposal practices did not result in significant degradation of ground water quality.

Surface water monitoring at five sites indicated no discernible impact or only minimal impact from the waste management practices. Observed increases above background levels were primarily for total dissolved solids and sulfate.

The study also demonstrated the problems associated with monitoring and evaluating ground water impacts caused by large waste disposal facilities situated in complex hydrogeological settings. In addition, difficulities associated with discerning impacts from sources containing constituents that occur naturally in the area were encountered.

Introduction

Section 8002 (f) and (p) of the Resource Conservation and Recovery Act of 1976 and the Solid Waste Disposal Act Amendments of 1980 require that the U.S. Environmental Protection Agency (EPA) conduct an investigation of solid waste management in the mining industry. Currently the mining industry is responsible for 40 percent, 2.3 billion tons, of all solid wastes generated annually in the United States.

In September 1979 PEDCo Environmental, Inc. (PEI) was awarded a contract (EPA Contract No. 68-01-2900) by the EPA Office of Research and Development to conduct an evaluation of management practices for mine solid waste storage, disposal, and treatment. PEI and its principal subcontractors, Geraghty & Miller, Inc. and Skelly and Loy, Inc., completed its investigation and submitted its findings in December 1983. The investigation included waste sampling at 65 sites and water quality monitoring at 8 sites. The study included mining and beneficiation of phosphate, copper, lead, zinc, gold, silver, iron, tungsten, and molybdenum and the mining of uranium.

Site Selection

The mining industry in the United States consists of over 12,000 facilities representing about 50 major segments. It was the objective of EPA to identify and more closely evaluate those mining segments, waste streams, and waste management practices that present the greatest potential for adverse environmental impact. Identification of these potential problem areas required that PEI develop a selection methodology to eliminate all other segments, waste streams, and practices.

The selection process was composed of three principal evaluation steps:

1. Assessment of mining industry segments
2. Assessment of waste streams and disposal methods
3. Determination of site typicality

Step 1 - Assess Industry Segments

Industry segments were evaluated using available published data with respect to (1) magnitude of waste generated; (2) waste characteristics based on corrosivity, toxicity, and radioactivity; and (3) potential to produce sediment loading to surface water, emission of fugitive dust, and negative effects on such areas as floodplains, wetlands and endangered species. All metal mining and beneficiation segments were evaluated. Coal mining waste and uranium mill tailings were not included in this study because these wastes are covered by the Surface Mining Control and Reclamation Act and the Uranium Mill Tailings Disposal Act, respectively.

Step 1 results identified the phosphate, copper, lead, zinc, gold, silver, molybdenum, tungsten, and iron mining and beneficiation segments, and the uranium mining segment for further evaluation. These industry segments were then subjected to the second level of assessment.

Step 2 - Assess Waste Streams and Disposal Methods

Sixty-five sites covering the mining segments identified in Step 1 were visited for the purpose of collecting additional information on waste characterization and waste management practices. The site visits included a sampling program to collect wastes from each of the waste management streams for chemical, physical, and radiological evaluation. The waste streams and practices were then evaluated based on (1) magnitude of waste generated; (2) waste characteristics based on EP Toxicity, corrosivity, radioactivity, and use of cyanide; (3) potential to produce sediment loading to surface water, fugitive dust, and location within environmentally sensitive areas; and (4) potential for transport of waste material and hazardous constituents into the environment based on particle size and the presence of water.

Nine specific waste streams and disposal methods were identified in Step 2 for further evaluation. They included uranium waste rock dumps and mine water ponds, phosphate clay slime ponds, sand tailings dumps and waste rock dumps, copper tailings ponds and leach dumps, lead/zinc tailings ponds, gold/silver tailings ponds, and molybdenum tailings ponds.

Upon selection of the waste practices, geographic locations and distribution were evaluated to assist selection of the most significant and representative sites for site assessment. This effort resulted in the selection of the following mining districts for further assessment:

- Wyoming uranium waste rock dumps
- Wyoming and New Mexico uranium mine water ponds
- Florida phosphate clay slime ponds and sand tailings dumps
- Florida and Idaho phosphate mine waste rock dumps
- Southwest copper leach dumps

- Southwest copper tailings ponds
- Missouri and Tennessee lead/zinc tailings ponds
- Nevada and South Dakota gold/silver tailings ponds
- Colorado molybdenum tailings ponds

Several mines from the original 65 included in these districts were revisited for the final (Step 3) evaluation.

Step 3 - Determine Study Sites

To choose sites that are typical or most representative of the industry segments and waste management practices, the individual sites were assessed for typicality based upon (1) waste characteristics; (2) environmental condition based on climate and hydrogeology; (3) presence of operational monitoring systems and age; and (4) external influence which could mask the environmental effects. Eight specific practices were selected for field evaluation:

- New Mexico uranium mine water ponds
- Florida phosphate clay slime ponds and sand tailings dumps
- Idaho phosphate mine waste rock dumps
- Southwest copper leach dumps
- Southwest copper tailings ponds
- Missouri lead/zinc tailings ponds
- Nevada gold/silver tailings pond
- South Dakota gold/silver tailings pond

Monitoring Program

Between September 1981 and February 1982 ground water and surface water monitoring stations were installed at each of the eight selected study sites (Table 1). Ground water conditions were evaluated at seven of the eight sites and surface water at five. Ground water was not monitored at the Idaho phosphate site because the practice extended over a ground water divide into different basins; surface water was not monitored at the New Mexico uranium site or either southwest copper site because surface water was not perennially present.

Surface water and ground water samples were collected on five separate occasions over a 6- to 9-month period. The last water quality samples were collected in August 1982. Collected samples were analyzed for an extensive list of physical, chemical, and radiological constituents (Table 2).

TABLE 1. MONITORING PROGRAMS CONDUCTED AT THE SELECTED SITES

Site	Media monitored and number of monitoring stations	
	Ground water	Surface water
South Dakota Gold/Silver Tailings Pond	7 wells	4 stations
New Mexico Uranium Mine Water Ponds	8 wells	None
Idaho Phosphate Mine Waste Rock	None	3 stations
Southwest Copper Tailings Pond	6 wells	None
Nevada Gold/Silver Tailings Pond	5 wells	3 stations
Missouri Lead/Zinc Tailings Pond	4 wells	2 stations
Southwest Copper Leach Dump	6 wells	None
Florida Phosphate Clay Slimes Pond/Sand Tailings Disposal Area	12 wells	3 stations

TABLE 2. ANALYTICAL PARAMETERS

Acidity[a]
Alkalinity[a]
Ammonia
Antimony
Arsenic
Barium
Beryllium
Cadmium
Calcium
Chloride
Chromium
Conductivity[a]
Copper
Cyanide (total and easily dissociable)
Fluoride
Iron
Lead
Magnesium
Manganese
Mercury

Molybdenum
Nickel
Nitrate
pH[a]
Phosphorus
Potassium
Selenium
Settable solids
Silver
Sodium
Sulfate
Suspended solids
Tellurium
Tin
Temperature[a]
Total dissolved solids
Total dissolved carbon
Turbidity
Vanadium
Zinc

Radionuclides

^{228}Ac
^{212}Bi
^{214}Bi
^{210}Pb
^{210}Po
^{226}Ra

^{232}Th
^{235}U
^{238}U
^{234}U
Gross Alpha
Gross Beta

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[a] Field analysis.

The surface water monitoring program included upstream and downstream sampling stations. Each station consisted of a 60° V-notch wier for low-flow determination and a rectangular sharp-crest wier for high-flow determination. A crest gauge was installed to record high water between individual sampling periods. An automatic sampler and flow recorder were used during the five individual 2-day sampling periods. At some sites grab samples were collected instead of the composites produced by the automatic samples because of low flow or extreme cold conditions.

The ground water monitoring program was designed to monitor the water-table aquifer at each site. Monitoring wells were placed at locations to evaluate water quality hydraulically upgradient and downgradient of the practice being evaluated. The site-specific ground water monitoring programs consisted of one to two upgradient wells and four to six downgradient wells. Two-inch, polyvinyl chloride (PVC) threaded flush joint monitor wells were used everywhere except at the southwest copper tailings pond site where 4-inch compression coupling PVC monitoring wells were used. Bores were constructed using air rotary or cable tool drilling except at the Florida phosphate site where standard rotary wash drilling was used. Water samples were collected using a bottom filling PVC bailer or peristaltic pump equipped with Tygon tubing except at the southwest copper tailings pond site where it was necessary to install submersible pumps because of the great depth to ground water. At each site two or more wells were tested to determine permeability.

The collected water-quality data were evaluated to identify impacts to the environment. The evaluation was based on the chemical analyses of samples collected to characterize the wastes and samples from the ground water and surface water monitoring stations to identify trends indicative of potential impacts. Specifically two tests were applied which involved (1) a comparison of water quality downgradient of the practice with water quality upgradient of the site and (2) comparison of selected tracer constituents from the waste with samples downgradient of the site. The Student's t-test evaluation for significance was applied for individual parameters at the 95 percent confidence level. Ground water quality data were also evaluated using a multi-element cluster analysis. The site evaluation also included definition of the ground water flow system based on water level determinations made in each well at the site.

Study Results

Even though this investigation included two gold, two copper, and two phosphate sites and one lead/zinc and one uranium site, the objectives of the study were not directed toward mining segments as much as toward waste management practices. As such, this investigation examined four tailings ponds, and one each mine water pond, leach dump, overburden pile, clay slimes pond, and sand tailings disposal area.

Effects on ground water quality were identified at each of the four tailings pond sites (Southwest copper, Missouri lead/zinc, Nevada gold/silver, and South Dakota gold/silver). Chloride, sulfate, and/or total dissolved solids increases in monitoring wells were attributable to the practice in all cases. Additionally cyanide was present in ground water near the two gold tailings ponds. At all four sites the practices did not cause an increase in metals considered to be hazardous by EPA. Effects were observed in only a few wells at each site. Impacted wells were sited in close proximity to a water table mound produced by the practice or in areas of highly fractured bedrock that appeared to act as a preferential flow path.

Surface water effects were detected at the Nevada gold/silver tailings pond. The effect was attributed to increased arsenic, manganese, fluoride, and total dissolved solids at the site. Cyanide was not identified in surface water samples. A statistically significant increase in several parameters (calcium, sodium, potassium, fluoride, magnesium, nitrate, and sulfate) was detected at the Missouri lead/zinc and South Dakota gold/silver tailings ponds. The increase could not be confirmed to be caused by the practice. Surface water was not monitored at the southwest copper tailings pond site.

Ground water effects at the New Mexico uranium mine water ponds were similar to those identified at tailings ponds inasmuch as chloride, sulfate, and total dissolved solids concentrations were increased as a result of the practice. A statistically significant increase in gross Beta was also identified in two wells. Increased TOC, nitrate, calcium, and magnesium concentrations in two wells could not be attributed to the mine water ponds. Surface water was not monitored at this site.

Nine parameters identified as potential contaminant indicators from the southwest copper leach dump site were found in the well originally intended to represent background conditions at levels that indicate contamination. These parameters included calcium, manganese, nickel, zinc, pH, total dissolved solids, sulfate, and fluoride. One downgradient well produced samples having calcium, total dissolved solids, and sulfate levels attributable to the leach dump practice. The background well is believed to have been placed at a location that exhibits reversed ground water gradient because of mine dewatering approximately one-half mile away. The one affected downgradient well is located within a draw that appears to serve as a zone of preferential ground water flow. Surface water was not monitored at this site.

No parameters identified as potential indicators of seepage from the Idaho phosphate overburden dump were identified at increased concentrations in surface water downstream of the waste management practice. Ground water was not monitored at this site due to monitoring and data interpretation problems that could have been encountered as a result of the waste material being placed in two drainage basins.

Two different waste management practices were evaluated at the Florida phosphate site: clay slime ponds and sand tailings disposal areas. Additionally two different aquifers were monitored: the water table and Upper Floridan. Increased concentrations of chloride, fluoride, total dissolved solids, total phosphorous, and total organic carbon in the water table aquifer were attributed to the sand tailings disposal area. The levels identified were only slightly above ambient conditions. Collected data do not indicate that the quality of water in the water table aquifer has been affected by the clay slimes ponds. No effect from either practice was observed in the Upper Floridan Aquifer, and no impacts could be discerned from either practice in surface water at the site.

The objective of the investigation was to select and evaluate those waste management practices having the greatest potential for environmental damage. While all practices except the Idaho phosphate practice were found to release contaminants, few practices were found to present a great potential for damage. Only one well at the southwest copper leach dump yielded water that could be considered environmentally threatening because of a low pH and high metals contents. Concentrations of cyanide identified in samples from one well at the Nevada gold/silver tailings pond and two wells at the South Dakota gold/silver tailings ponds were found to fluctuate significantly. Cyanide concentrations, in general, were at or slightly above the laboratory detection limit, however. All impacted wells exhibited increased concentrations of such parameters as chloride, sulfate, calcium, and total dissolved solids.

Monitoring Difficulties

During the design of the ground water monitoring systems, two key factors influenced the location of monitoring wells. One required that all monitoring wells be installed in the water table aquifer (except at the Florida phosphate site where a deeper artesian aquifer was also monitored), and the second required that the background well be located hydraulically upgradient from the practice. Because of the very large size of some of the practices monitored (over one square mile), rugged terrain, and geologic complexity, it was found that at five sites (New Mexico uranium, South Dakota gold/silver, Nevada gold/silver, and both southwest copper sites) the background well was installed into a water table aquifer having a different lithology than the water table aquifer at the downgradient location. While it could be demonstrated that the background monitoring point was hydraulically upgradient of the practice, it was found that the quality of water produced was not reflective of background conditions for the unit being monitored at the downgradient point.

Statistical analyses of the collected water quality data using the Student's t-test method identified numerous parameters at statistically higher or lower levels in downgradient wells. The causes could not be clearly attributed to the practice being monitored because the waste quality did not reflect the observed downgradient quality. Further, the principal indicator contaminants from all practices were identified to represent the major rock forming elements and complexes (i.e., sodium, calcium, chloride, sulfate); these were the same constituents of the ground water samples affected by differing lithology at well locations.

To offset these problems, it was necessary to define ambient or background conditions in the downgradient aquifer via other means. This included using published information where available or selection of the best quality downgradient well for comparison with other wells. In all cases differing quality between a downgradient well and ambient conditions was not determined to be the result of the practice unless three or more indicator parameters of the practice were identified at increased (or pH decreased) concentrations in the downgradient monitoring well.

In two cases (southwest copper leach dump and Missouri lead/zinc tailings pond) background wells were found to be affected by the practice because the wells were located within the zone of influence of the practice. Natural conditions at the southwest copper site and the legal access limitations at the Missouri lead/zinc site prevented location of these wells at points that would represent unaffected background conditions.

Acknowledgment

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Bibliographies

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Robert L. Hoye is an Environmental Scientist with six years experience in the evaluation of impacts from waste disposal practices. He holds an M.S. degree in Environmental Engineering from the University of Cincinnati. Mr. Hoye was PEDCo's Assistant Project Manager during completion of this project and was a principal investigator and primary author of the project reports. He has developed and implemented waste characerization studies at over 100 sites across the country and is currently Project Manager of hydrogeological investigations at two mine waste disposal sites and waste characterizations at 24 smelting/refining facilities.

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Session IV

Dealing with Agricultural Chemicals and Land Application Practices

EFFECT OF ARIZONA'S 1980 GROUND WATER CODE ON THE PREVENTION OF GROUND WATER DEGRADATION FROM AGRICULTURAL PRACTICES

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Abstract

In 1980, the Arizona legislature enacted ground water management legislation which established a system of regulation and management of ground water use in certain areas of the State. Active Management Areas (AMAs) were created in four ground water basins exhibiting severe ground water depletion. The goal of three of the AMAs is to attain safe yield by the year 2025. To accomplish this, the AMA's must develop and implement management plans every ten years with increasingly restrictive water conservation conditions. The fourth AMA is located in an agricultural area, and was established to preserve the existing agricultural economy for as long as feasible. Three "Irrigation Non-Expansion Areas" (INAs) were also created; these require lesser restrictions. New irrigation agricultural land is prohibited in both AMAs and INAs, and only lands irrigated between 1975 and 1979 can now be irrigated.

The three most common types of pollutants associated with agricultural irrigation are nitrates, total dissolved solids, and pesticides. Nitrates are the residual of fertilizer application. Historically, nitrates are percolated to ground water not necessarily through overfertilization, but through overirrigation. Elevated dissolved solids are the residual of poor quality irrigation water combined with the leaching of constituents in the vadose zone, and have also percolated to ground water through overirrigation. The practice of overirrigation, which may result in ground water degradation, will diminish as a result of the strict regulation of ground water withdrawal in the AMAs. In addition, the restriction of irrigation in the INAs will result in limiting the annual quantity of deep percolation. The agricultural industry is aware that, to meet future regulatory requirements, higher irrigation efficiencies will have to be achieved. What the industry may not be aware of is that attaining high irrigation efficiencies is also a Best Management Practice for preventing ground water degradation.

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Introduction

Agricultural irrigation is one of the classical sources of non-point source pollution. Specifically, agricultural irrigation consists of the diffuse application of irrigation water, fertilizers and other soil amendments, and pesticides and herbicides over vast areas of cultivated land. Needless to say, residuum from this application may, and in many cases, has, percolated to ground water. The mechanics of ground water degradation from agricultural irrigation are discussed.

The Arizona Department of Health Services is empowered by statute to manage ground water quality throughout the State; this is discussed in detail in Gordon et al. (1983). The Arizona Department of Water Resources is responsible for managing ground water quantity through the 1980 Ground Water Code (Briggs, 1983). The Code is discussed below with special reference to irrigated agriculture, which is intensely regulated from a water conservation standpoint. The issue has arisen in Arizona as to how agricultural irrigation should be regulated with respect to ground water quality. The purpose of this paper is to address this issue, describing how ground water degradation from agricultural irrigation occurs, and what effect the 1980 Ground Water Code will have on the deep percolation of irrigation water.

Ground Water Degradation From Irrigation Practices

Salt and Nitrate

Water for irrigation may be applied through subirrigation (drip systems) sprinklers, or surface (furrow or border) systems. Most cultivated land in Arizona is irrigated by some type of surface system. Fields are graded such that water applied via ditches at the top of the field will flow across the field. Irrigation water leaves the field in one of three ways:

1) Evapotranspiration. This includes the water which is used by the plant and transpired through its leaves, and water which is evaporated from the soil.
2) Surface Runoff. Water runs off the lower portion of the field when applied at a rate greater than the infiltration rate. Some surface runoff may be necessary to assure that lower portions of the field receive adequate water.
3) Deep Percolation. This is water that seeps through the root zone and can no longer be captured by the plant. This water eventually percolates down to the ground water table.

The amount of water used for evapotranspiration is the consumptive use, as this water cannot be recovered. Surface runoff is easily collected and reused, either through application on fields at lower elevations, or by pumping back to the top of the field for reapplication. Deep percolation water is also reused, since it eventually recharges ground water and can be recovered by additional ground water pumping. The ratio of the quantity of consumptively used water to the total quantity of water applied is the irrigation efficiency (E_i):

$$Ei = \frac{\text{consumptive use quantity} \times 100}{\text{quantity of water applied}}$$

The efficiencies of surface irrigation systems vary from 50 to 90 percent depending on how well the system is designed and managed (Bouwer, 1983).

Although deep percolation water is returned to the ground water supply to be reused, it is usually of a much lower quality than applied water. This reduction in quality results from two processes:

1) Concentration of salts. Water which is evapotranspired is essentially pure; therefore, salts dissolved in the applied irrigation water are concentrated in the water remaining in the soil.
2) Leaching of chemicals applied to or present in the soil. These chemicals include applied nitrogen fertilizers and pesticides, as well as natural salts present in the soil.

The concentration of salts (total dissolved solids) in deep percolation water from applied irrigation water can be approximated from the equation:

$$C_i \, D_i = C_{dp} \, D_{dp}$$

where C_i = Concentration of salt in applied irrigation water.
D_i = Amount (depth) of irrigation water applied.
C_{dp} = Concentration of salt in deep percolation water.
D_{dp} = Amount (depth) of deep percolation water.

A rearrangement of the above equation

$$C_{dp} = C_i \, D_i / D_{dp}$$

shows that as D_{dp} decreases with respect to D_i (increased irrigation efficiency) C_{dp} must increase. Such a decrease in the quantity of deep percolation water is expected as irrigators move towards more efficient irrigation. Although the concentration of salt (C_{dp}) in deep percolation water will increase, the total salt load will probably decrease. For example, if a crop requires four feet of water for evapotranspiration and six feet of water containing 800 mg/l TDS (C_i = 800 mg/l) is applied (D_i = 6 ft), then (assuming no surface runoff) two feet of water will go towards deep percolation (D_{dp} = 2 ft). This represents an irrigation efficiency of 66 percent. Therefore, the TDS concentration of the deep percolation water is represented by:

$$C_{dp} = (800 \text{ mg/l}) \, (6/2) = 2400 \text{ mg/l}.$$

For one acre of this crop, the additional salt load to ground water would be:

$$(2400 \text{ mg/l}) \, (.264 \text{ l/gal}) \, (325{,}851 \text{ gal/ac-ft}) \, (2 \text{ ac-ft}) = 412{,}918{,}380 \text{ mg or almost 413 kg.}$$

If only five feet of similar quality water is applied (D_i = 5 ft), the amount 4
of deep percolation water (again assuming no surface runoff) would be one foot (D_{dp} = 1 ft) and C_{dp} would equal:

(800 mg/l) (5/1) or 4,000 mg/l.

This case represents an irrigation efficiency of 80 percent. The salt load for one acre of this crop would be:

(4000 mg/l) (.264 l/gal) (325,851 gal/ac-ft) (1 ac-ft)
= 344,098,650 mg or about 344 kg.

Therefore, although increasing the irrigation efficiency in this example results in an increased concentration of TDS, italso results in a total decrease in the quantity of salts entering ground water.

As water moves through the vadose zone to the ground water table it equilibrates, dissolving or precipitating salts. Precipitation of salts, primarily calcium carbonate (calcite) and calcium sulfate (gypsum), is favored by more concentrated solutions. As an increase in irrigation efficiency will increase the salt concentration in deep percolation water, the tendency for salts to precipitate prior to entering the ground water table is increased.

Water percolating through the soil will leach nitrate derived from nitrogen fertilizers. The greater the amount of deep percolation water, the greater the amount of nitrate that may be leached from the root zone. Once this nitrate-rich water moves below the root zone there is little opportunity for removal of nitrate.

Deep percolation also affects the rate at which contaminants travel through the vadose zone. Unsaturated hydraulic conductivity is directly proportional to water content. In unsaturated material, part of the pore channels are filled with air, which physically obstructs water movement. Water then flows only through the finer pores, which may be saturated, or in films around the soil particles. Therefore, when deep percolation takes place, the water content in the vadose zone increases, increasing the unsaturated hydraulic conductivity.

Increased salt and nitrate values have been observed in ground water below irrigated agricultural areas. Ground water near the town of Gilbert, Arizona was found to have salt contents (TDS) ranging from 830 to 2190 mg/l. The nitrate contents ranged from 32 to 60 mg/l (Schmidt, 1983). This is in contrast to the quality of applied irrigation water; irrigation water is supplied by canals which deliver a combination of both surface and ground water. The average concentrations of TDS and nitrate in the canal water is 500 mg/l and 5 mg/l, respectively.

Pesticides

For some time it was assumed that pesticides with high molecular weights would not reach ground water because of their low solubilities in water and tendency to adsorb to clay and organic matter in soil. Light-molecular weight pesticides were assumed to return to the atmosphere through volatilization (Bouwer, 1978). However, in recent years a number of pesticides have been identified in ground water. In Fresno, California, 1,2-dibromo-3-chloropropane (DBCP) was found in ground water at depths up to 200 ft. deep (Schmidt, 1983). The pesticide DBCP has also been found in ground water in Yuma and Mesa, Arizona. In the Mesa area, DBCP was thought to result from a "short-circuiting" mechanism rather than through deep percolation (Filleman and Nemecek, 1980). Short-circuiting mechanisms include inadequate surface seals around wells and cross contamination in the subsurface due to improperly drilled or constructed wells.

Arizona's Ground Water Code

Primarily in response to massive overdraft problems in the central and sourthern part of the State, Arizona adopted a comprehensive ground water code that restricts withdrawal of ground water, encourages conservation practices, and provides for some ground water quality management.

The code delineates four large Active Management Areas (AMA), and three Irrigation Non-Expansion Areas (INA) where agricultural uses of water predominate. These AMA's and INA's are shown on Figure 1.

The Arizona Department of Water Resources (ADWR) is responsible for enforcing the provisions of the code, and will promulgate five Management Plans in the next 45 years to achieve the objectives of the code. Three of the AMA's, Phoenix, Prescott and Tucson, and under legislative mandate to achieve safe-yield by the year 2025. The fourth AMA, Pinal, must sustain the existing agricultural economy while allowing development of non-irrigation uses.

As their names imply, each management area either actively manages ground water withdrawals and applications of ground water to crops, or limits the expansion of land under cultivation. In AMA's, ground water withdrawals are controlled by a system of rights and permits, and application rates are controlled by setting irrigation water duties for agricultural lands. No new agricultural lands are permitted. In INA's no controls on withdrawals are enforced, but new agricultural lands cannot be developed.

Each farm unit within an AMA will be allocated a maximum annual ground water allotment. This water allotment applies to farmers who use either ground water alone or a combination of ground water and surface water. The allotment is calculated in the following manner:

$$\text{Maximum Annual Ground Water Allotment} = \text{Irrigation Water Duty} \times \text{Water Duty Acres}$$

where

$$\text{Irrigation Water Duty} = \frac{\text{Total Irrigation Requirement/Total Planted Acres}}{\text{Assigned Irrigation Efficiency}}$$

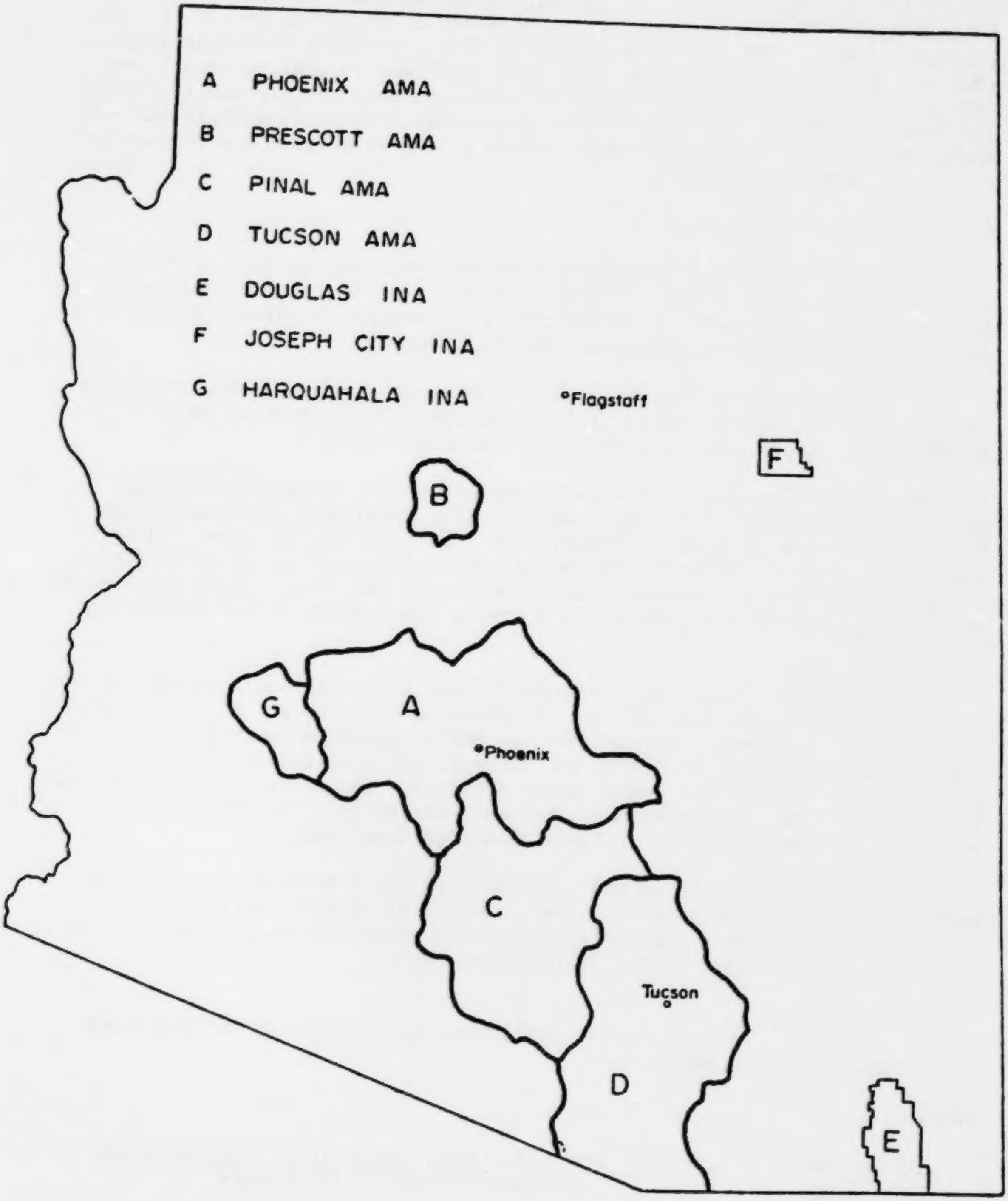
A PHOENIX AMA
B PRESCOTT AMA
C PINAL AMA
D TUCSON AMA
E DOUGLAS INA
F JOSEPH CITY INA
G HARQUAHALA INA
Flagstaff
F
B
G
A
Phoenix
C
Tucson
D
E

Figure 1. Active management areas and irrigation non-expansion areas in Arizona.

Total planted acres are the sum of the acres planted during the period 1975 through 1979, but do not include double-cropped acres. Water duty acres are the highest number of acres in the farm that were legally irrigated during any one year from 1975 through 1979. Double-cropped acres are counted only once; the irrigation water duty calculation includes the additional irrigation requirement for the double-cropped acreage.

The irrigation water duty is defined as:

> "The quantity of water reasonably required to irrigate the crops historically grown in a farm unit and shall assume conservation methods being used in the State which would be reasonable for the farm unit including lined ditches, pump-back systems, land leveling and efficient application practices, but not including a change from flood irrigation to drip irrigation or sprinkler irrigation" (A.R.S. 45-564).

The total irrigation requirement equals the average amount of water that must be applied annually to satisfy the consumptive use requirements of the crops historically grown, plus other water needs of the plants (e.g., germination) and any leaching requirement. The irrigation requirement for each farm unit was based on the cropping history submitted by ground water rights applicants and on reliable consumptive use values for each crop.

Irrigation efficiency indicates the general effectiveness of water application and is a function of many variables including evaporation loss, soil intake rate, land slope, and plant consumptive use requriements. ADWR will assign an irrigation efficiency to each farm unit in each AMA. The average assigned irrigation efficiency in the Phoenix AMA, for example, will fall between 55 and 85 percent. In each subsequent management period, the assigned irrigation efficiency will be increased, resulting in a subsequent decrease in deep percolation.

There are several other provisions in the Ground Water Code that will directly effect water quality or can be used to restore water quality in the major aquifers of the State. These provisions are listed below:

1) Prevention of contamination from the surface or cross contamination in the subsurface through enforcement of well drilling and construction regulations;
2) Management of poor quality ground water areas or contamination problems through administration of ground water withdrawal rights and permits;
3) Selective augmentation of ground water supplies through recharge projects; and
4) Comprehensive ground water quality management plans developed and administered in cooperation with the Arizona Department of Health Services.

The development and administration of coordinated ground water quality management plans is discussed in Briggs (1982) and Gordon et al. (1983).

Discussion

It has been demonstrated in this paper and by others that the practice of overirrigation, resulting in deep percolation, may, and in many cases has, resulted in ground water degradation. Reduction of deep percolation through the establishment of water duties and conservation requirements may have the most dramatic long term effect on ground water quality beneath irrigated lands.

The restriction of irrigation in the INA's will result in limiting the annual quantity of deep percolation. Increasing irrigation efficiencies and reducing irrigation application rates will reduce the amount of water available for deep percolation to the water table. Aside from reducing permeability in the unsaturated zone because of lowered moisture content, the vertical velocity of chemical consituents such as nitrates and pesticides will be retarded. This will give pesticides more time to degrade. Displacement of original soil water will be minimized (Bouwer, 1978), and leaching of native salts at depth will also be reduced. The agricultural industry is aware that, to meet future regulatory requirements, higher irrigation efficiencies will have to be achieved. What the industry may not be aware of is that attaining high irrigation efficiencies is also a Best Management Practice for preventing ground water degradation.

To prevent the "short-circuiting" of pesticides, as well as any other pollutants, ADWR well drilling and construction regulations have been developed. Well construction standards for new wells for the prevention of ground water degradation include the disallowance of perforations above the water table, a required 19 foot surface seal consisting of steel casing and cement grout, and the required casing and/or cementing of water-bearing units containing mineralized or polluted water to prevent cross contamination in the subsurface. For existing wells, if an inner casing is installed to prevent leakage of undesirable water into a well, the space between the two well casings shall be completely sealed to prevent the movement of water between the two casings. Requirements for well abandonment include the filling or sealing of the well so as to prevent the well, including the annular space outside the casing, from being a channel allowing the vertical movement of water. And finally, the Director of ADWR may require that further additional measures be taken if it is determined that the literal application of the well construction standards would not adequately protect the aquifer or other water users.

Summary

1. The amount of water evapotranspired by crops is the consumptive use, as this water cannot be recovered. The ratio of the quantity of consumptively used water to the total quantity of water applied is the irrigation efficiency. Deep percolation is inversely proportional to the irrigation efficiency.
2. Deep percolation water is usually of a much lower quality than applied water. This results from the concentration of salts from applied irrigation water through evaportranspiration, and the leaching of chemicals applied to or naturally present in the soil. Deep percolation also increases the water content in the vadose zone, increasing the

unsaturated hydraulic conductivity and thus, the rate at which contaminated water seeps to ground water.

3. The 1980 Ground Water Code was created in response to massive overdraft problems in portions of Arizona. The Act created four Active Management Areas. Each farm unit within an AMA will be allocated a maximum annual ground water allotment based upon, among other variables, an assigned irrigation efficiency. In subsequent management periods, the assigned irrigation efficiency will be increased, resulting in a subsequent decrease in deep percolation.
4. The practice of overirrigation, resulting in deep percolation, may, and in many cases has, resulted in ground water degradation. Reduction of deep percolation through the establishment of regulatory conservation requirements may have the most dramatic long term effect on ground water quality beneath irrigated lands. Attaining high irrigation efficiencies is a Best Management Practice for preventing ground water degradation.
5. Pesticide contamination may be caused by "short-circuiting" mechansisms such as inadequate surface seals around wells and cross contamination in the subsurface due to improperly drilled or constructed wells. To prevent the short-circuiting of pesticides, as well as any other pollutants, ADWR well drilling and construction regulations have been developed.

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A RAPID ASSESSMENT METHODOLOGY FOR LEACHING OF AGRICULTURAL CHEMICALS

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Introduction

The growing threat to ground water supplies from chemicals introduced into the environment is an issue of national concern. This concern is reflected by the current proposed guidelines for the registration of pesticides (U.S. EPA, 1978), which requires that a pesticide's mobility in soils be established. The potential for a pesticide to leach through the soil profile can indeed be measured in laboratory or column studies. In general, however, such studies can only establish the tendency for a compound to leach relative to a reference compound/soil combination. Determination of whether the compound will actually leach *significantly* under *field* conditions requires quite a different approach.

The movement of a pesticide through the soil in an agricultural application is a function of soil physical and chemical properties, pesticide properties, local watershed and meteorological conditions, and, additionally, soil, water and pesticide management practices. Field or plot studies unfortunately require a great deal of time and expense to answer the important questions concerning pesticide mobility. The key question is:

> Does the use of a pesticide in a given agricultural situation constitute a significant environmental risk with regard to ground water contamination?

Fortunately, computer models which simulate the movement of pesticide in soils have been developed and can be applied with less expense and in far less time than field programs. Even so, with the extremely large number of chemical products and multiple soils, watershed and management combinations, the application of models in every case may not be justifiable or feasible.

In many cases, detailed analyses can be eliminated through the use of prior knowledge of the system involved. In other words, the pesticide/soil/crop/management variable combination could be screened to determine if leaching to ground water occurs, and, if it does, to determine its frequency and severity (i.e. how much, how often). Such information would be extremely important in eliminating unnecessary use of resources for laboratory, field and modeling studies. To be useful, the procedure must be quickly and easily performed.

This manuscript provides a summary of such a screening methodology developed for the U.S. Environmental Protection Agency (Dean, et al., 1984) known as LEACH (Leaching Evaluation of Agricultural Chemicals Handbook).

The purpose of this study was to provide a methodology by which new agricultural chemicals, or chemicals being considered for alternative uses can be readily screened to determine if they will move in significant quantities past the crop root zone. If significant leaching is predicted, the methodology was to provide a measure of its frequency and severity, and/or indicate the necessity for more detailed analyses. The method was to integrate the effects of climate, soil properties, pesticide properties and management practices on pesticide leaching.

In this methodology, "significant leaching" is considered to be any movement ($>0.05\%$ of the annually applied chemical mass) of non-degraded pesticide (i.e., toxic residue) past the rooting depth of the crop. This definition is both necessary and convenient in terms of methodology development because it allows the methods to ignore the complicating effects of local ground water conditions on the fate of the pesticide. In order to perform a screening assessment of this type on a national scale, such local effects could not be feasibly considered. The rooting depth is a convenient datum because many pesticide attenuation processes, such as plant uptake, cease below this point and others, such as microbial degradation and soil adsorption may be less prominent. Therefore, below this depth, the pesticide can be considered, in a truer sense, more "conservative" than at some merely arbitrary datum in the soil profile. The adoption of this definition is inconvenient, however, to the user. The results alone cannot and should not be interpreted as an assessment of exposure to biological species or as data to compare to water quality criteria because the fate and transport processes in ground and/or adjacent surface waters have not been considered.

This project was designed to identify pesticide/soil/crop/management combinations conducive to pesticide leachate production and quantify their effects on a national scale. At this level it is entirely possible, even probable, that certain critical areas which may be identifiable on a more localized scale have been omitted.

Because of the diversity of pesticide, soils, crop, climatic and managerial scenarios possible, some combinations were eliminated at the outset of the project in order to develop a feasible project scope. The following sections discuss these limitations in each of four categories: pesticides, soils, management and crops.

Pesticide Properties

The methodology is limited to those pesticides which are hydrophobically adsorbed to soil materials according to linear and coincident adsorption/desorption isotherms. Pesticides which undergo significant (>10%) ionization at normal soil pH levels should not be screened. This percentage can be easily determined (Mills, et al., 1982). The methods are designed only for a single pesticide in a soil/water (two) phase system. They are not appropriate to pesticides dissolved in more than one solvent (e.g., water and toluene). Also, interactive effects (if such effects exist) of more than one pesticide are not considered. Neither are the methods generally applicable to pesticides which degrade to toxic daughter products with different decay and sorption characteristics from the parent.

Soil Properties

The soils are assumed to be relatively homogeneous in the horizontal and vertical directions. Soils should not have clay or plow pans, or any low permeability layers in the profile which would cause perched saturated conditions and/or lateral flow. Soils should be well dispersed, well drained and not subject to fracturing, cracking, swelling or shrinking. For these types of soils, results may be substantially in error.

Management Practices

The effect of management practices can only be evaluated to the extent that appropriate model parameter adjustments that describe their impacts can be estimated. In this methodology, the SCS runoff curve number technique is used to link management practices to infiltration, which is the driving force for pesticide leaching. Therefore, management scenarios capable of being treated are those which can be evaluated by this technique. These are, in general, limited to agricultural land use, tillage techniques and residue management. 4.

Effects of non-structural management practices such as pesticide formulation, application mode and timing, or foliar versus soil application are not directly considered.

Crops

The crops specifically considered are corn, soybeans, wheat, and cotton. Thus, the geographic areas covered are where these crops are grown in significant quantities. Use of this methodology for other crops outside these geographical areas is not recommended. Because the effects of irrigation were not considered in developing the methodology, it does not take into account this agricultural practice. Irrigation may or may not result in greater quantities being transported through the soil profile. Artificially drained land may be considered as long as drains are placed below the crop rooting depth and rooting depths correspond to those used in model simulations.

Methodology Development

This methodology was developed through the use of a pesticide leaching model that could integrate pesticide, climate, crop and soil characteristics, and management practices in order to generate time series of pesticide leached past the crop root zone. Several candidate models were investigated and PRZM (the Pesticide Root Zone Model), developed by the U.S. EPA, Athens Environmental Research Laboratory, was chosen (Carsel, et al., 1984). Sensitivity analyses were performed using the model to evaluate and quantify the impact of parameters most important to the leaching process. Then, using climatic, crop, soils, and pesticide usage information, nineteen (19) sites across the U.S. were delineated which are relatively homogeneous with respect to these characteristics. Twenty-five years of meteorologic data were subsequently used to generate pesticide leaching sequences at important levels of the key parameters as determined by model sensitivity analyses. This resulted in over 47,000 model simulation years. In order to display this information in a usable fashion, the annual leaching time series were organized into cumulative frequency distributions. With 1900 potential distributions, a matrix indexing system was devised to facilitate their organization and access. These matrices not only serve as tools to locate the frequency curves but also as screening tools for determining if significant leaching exists.

Description of Pesticide Leaching in Soils

There are two categories of parameters that affect the amount of pesticide that will leach below the crop root zone.

They will be referred to as transport parameters and supply parameters. Transport parameters affect the movement of the contaminant in the soil while supply parameters govern the quantity of contaminant present. Table 1 lists the transport and supply parameters important to PRZM, along with typical ranges of values for each. The range of values for each parameter is indicative of upper and lower limits found for agricultural soils and chemicals in current production.

The relative effect of each transport and supply parameter is governed by the mass balance/transport equation used in PRZM,

$$\frac{\partial C_w}{\partial t} = D \frac{\partial^2 (C_w/R)}{\partial z^2} - \frac{\partial (C_w V/R)}{\partial z} - (k_s + U) C_w \qquad (1)$$

where the transport parameters are:

R, the retardation factor
D, the dispersion/diffusion coefficient
V, the pore water velocity

and the supply parameters are:

k_s, the pesticide decay rate, and
U, the plant uptake rate.

C_w, is the dissolved pesticide concentration, t is time and z is depth in the soil profile. The retardation factor, R, is defined as:

$$R = 1 + \frac{K_d \rho_s}{\theta} \qquad (2)$$

in which K_d is the adsorption partition coefficient,
ρ_s is the soil bulk density and,
θ is the soil water content (volume basis).

The transport and supply parameters in Table 1 were used in model simulation sensitivity analyses to determine parameters to which pesticide leaching was most sensitive. Diffusion/dispersion was not included since it was felt that in soils in which leaching was important, advection would dominate transport.

For the analysis, 25 years of daily rainfall and evaporation data were obtained for Macon, GA. It was felt that this location, having high intensity rainfalls and sandy soils would show more pronounced sensitivity to other parameters. Infiltration depths were varied by manipulating runoff curve numbers from 51 to 94, encompassing the complete range of A to D soils planted in row crops or small grains (Schwab, et al., 1966).

TABLE 1. IMPORTANT PRZM PARAMETERS WHICH AFFECT PESTICIDE MOVEMENT IN SOILS

CATEGORY	PARAMETER	DEFINITION	UNITS	RANGE
Transport	R K_d	Adsorption Partition Coefficient	cm^3/g -soil	$0.0-10^3$
	ρ_s	Soil Bulk Density	g/cm^3	0.9-1.8 [1]
	V θ_{fc}	Field Capacity Water Content	cm^3/cm^3	4-26 [2]
	θ_{wp}	Wilting Point Water Content	cm^3/cm^3	1-17 [2]
	P	Precipitation	cm	50-150
	CN	Runoff Curve Number	-	51-94
Supply	TAPP	Pesticide Application Amount	Kg/ha	.25-20
	k_s	Pesticide Decay Rate	/day	$2.0 \times 10^{-4} - 10^{-1}$ [3]

[1] Baes and Sharp (1983)

[2] Rawls (1982)

[3] Nash (1980).

Not surprisingly, K_d was determined to be the most important value affecting pesticide leaching since it can take on such a wide range of values. 7.

Other findings relevant to the development of this methodology were that the percentage of applied pesticide leached below the root zone is independent of the quantity applied in PRZM. In the tests, 100, 1000, and 10,000 g/ha were applied and no differences were observed in the percentage leached. Consequently, a unit application rate was used in developing the methodology. In reality, adsorption/desorption isotherms for pesticides in soils can be non-linear. If this happens, then the percentage leached will not be independent of the loads. However, if concentrations in the soil are low (as they would be in agricultural applications) linearity can usually be assumed without introducing substantial error.

For the majority of cases, leaching of pesticide was shown not to be a potential problem. Only small values of R resulted in significant leachate production, while the entire range of observed k_s and CN values affected leaching. Regardless of hydrologic regime or half-life, values of R greater than 50 produced no significant leaching over a 25 year period in our simulations. This would eliminate concern over leaching for a majority of pesticides on the market today.

Two major considerations in the methodology development were to determine the least number of parameters that could be used to describe the leaching phenomenon and to select levels of these parameters to use for producing frequency information. From the results of the sensitivity analysis it was learned that, at a minimum, k_s, K_d, ρ_s, θ_{fc}, and CN must be considered. The remaining important parameters, precipitation, evapotranspiration, and depth of root zone were fixed by selection of location and crop type. Variations in rainfall amount and timing were factored in by using a long period of record at each site. Fortunately, ρ_s, K_d, and θ_{fc}, can be combined into R, the retardation factor, leaving only three parameters to vary.

Although the format and presentation of the frequency curves are simplified by the use of 3 parameters, all of the parameters involved in the transport equation can still be important. The production of leachate is most sensitive however to fluctuations in the retardation factor, curve number and decay rate and the consideration of these parameters is adequate for screening purposes. The actual ranges of model parameter values chosen to produce the frequency curves were distilled from those ranges reported in Table 1. Five values of R; 1, 3, 5, 10 and 50 and five values of k_s; 0.001, 0.005, 0.01, 0.05, 0.1 were chosen to represent the spectrum of problematic compounds. Four values of curve number appropriate to the soil/crop/management scenarios at each site were also used.

Representative Site Delineation

Leaching of pesticides using the four most economically important crops in the U.S. was investigated. These crops - wheat, corn, soybeans, and cotton - are grown extensively throughout the country under a wide range of soil characteristics, meteorologic regimes, and pesticide application scenarios. The most widespread pesticide leaching problems would likely arise in conjunction with usage on one of these crops.

The 19 representative sites were delineated by transferring agricultural, meteorologic, soils, and pesticide usage information to national maps. Each map contains the boundaries of a single characteristic of interest and was overlayed with other maps to discover naturally occurring spatial relationships. The major crop growing areas of the U.S. are delineated in the 1978 Census of Agriculture maps (U.S. Dept. of Commerce, 1982). Areas in which irrigation is essential for crop production were eliminated from further consideration. Maps of precipitation, soil nitrogen (as a surrogate for soil organic carbon), the SCS hydrologic soil groups, and major areas of herbicide and insecticide usage were also produced and overlayed. The final representative sites were drawn onto four maps, each corresponding to a single crop. These are displayed in Figures 1 to 4.

After the 19 sites were delineated, representative data were gathered for each in order to select inputs for PRZM. Table 2 lists meteorologic, soils, and crop data used which were used.

The meteorologic stations chosen to represent precipitation, evaporation, and temperature conditions at each site were selected from a data base developed by EPA-Athens staff using data provided by the National Center for Atmospheric Research. Chosen station records were required to contain a minimum of 25 years of continuous daily values, with as few missing entries as possible. Average annual rainfall for the chosen 25 year period was compared to rainfall isohyets in the representative site to check for compatability. If more than 25% of the evaporation data were missing, PRZM internally calculated daily values using Hamon's method. Missing temperature data were generated by averaging other values on that day over the 25 year period. This method was chosen, as opposed to interpolation, because many gaps extended longer than just a few days (sometimes a month or more data were missing). The meteorologic station numbers, periods of record, and average annual precipitation and temperature values for each site are also listed in Table 2.

SCS county soil surveys were used to select a representative soil type at each site. For each site three or four surveys were randomly selected, the soil textures listed, and

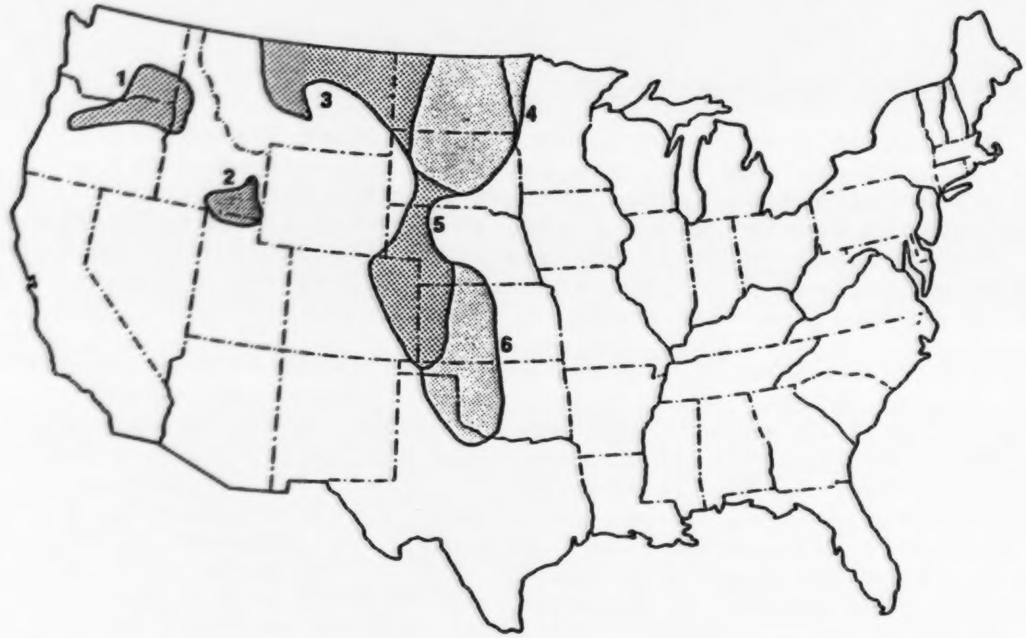

Figure 1. Representative Sites No. 1 - No. 6 Associated with Wheat Farms.

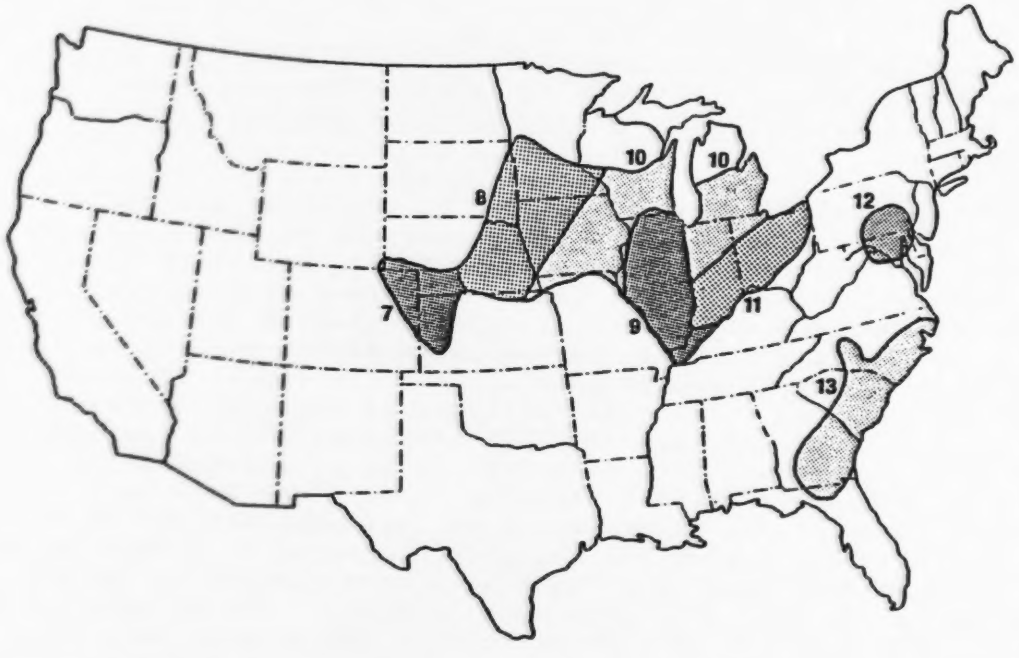
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Figure 2. Representative Sites No. 7 - No. 13 Associated with Corn Farms.

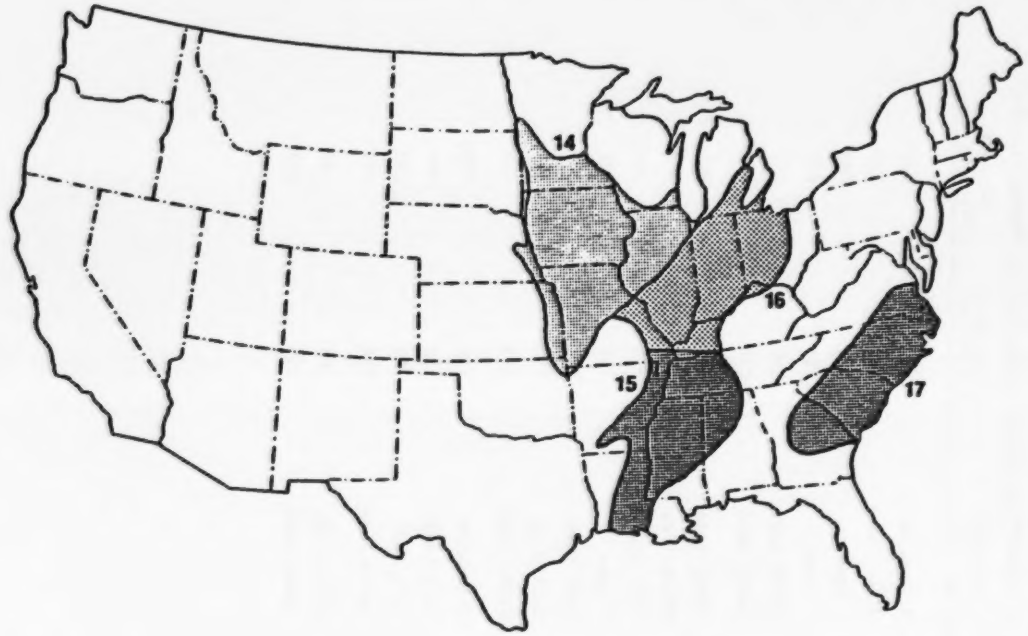

Figure 3. Representative Sites No. 14 - No. 17 Associated with Soybean Farms.

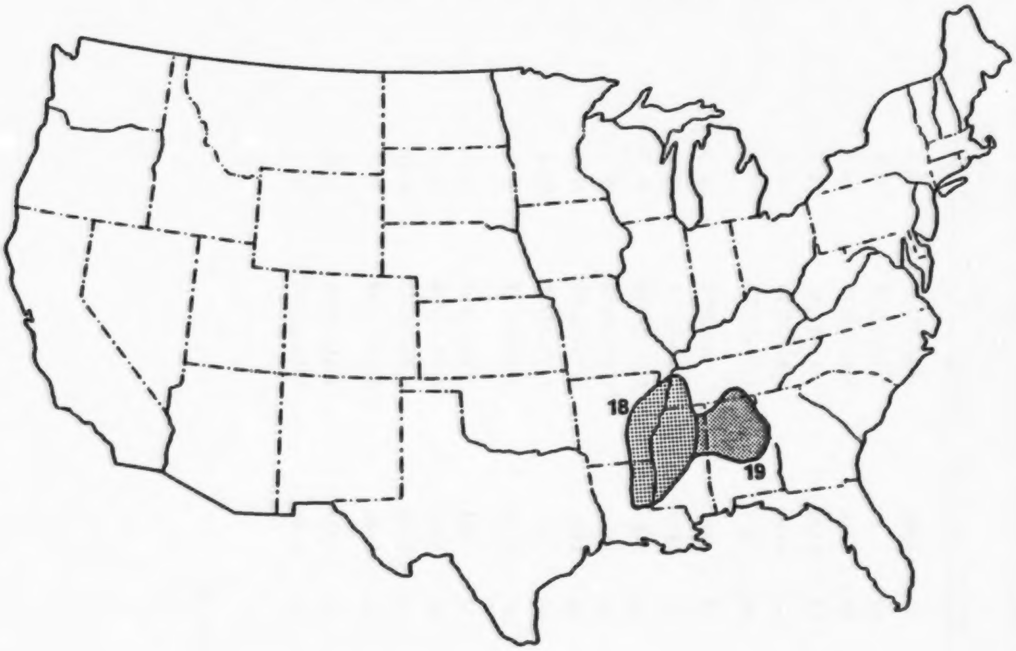

Figure 4. Representative Sites No. 18 - No. 19 Associated with Cotton Farms.

TABLE 2. SOILS, CROPS AND METEOROLOGICAL CHARACTERISTICS OF REPRESENTATIVE SITES

Site	State/Station #		Period of Meteorological Record	Annual Precip (in)	Annual Temp (°C)	Predominant Soil Type	Planting Day (Julian)	Harvest Day (Julian)
1	WA	8926	1/37-12/61	15.5	12.1	Silt Loam	79	227
2	ID	10	1/54-12/78	22.6	7.4	Silt Loam	125	237
3	MT	3110	1/54-12/78	11.6	7.6	Loam	121	232
4	ND	5479	1/48-12/72	15.6	6.9	Loam	121	227
5	NE	4455	1/54-12/78	18.2	10.8	Sandy Loam	262	215
6	KS	1383	1/50-12/74	20.9	12.1	Loamy Sand	262	215
7	CO	834	1/54-12/78	16.3	10.6	Loamy Sand	129	305
8	NE	3395	1/54-12/78	23.2	10.6	Silty Clay Loam	132	305
9	IL	8179	1/54-12/78	34.5	11.9	Silt Loam	136	305
10	MI	7690	1/54-12/78	34.6	9.4	Loam	136	275
11	OH	1466	1/54-12/78	36.1	9.2	Silt Loam	139	305
12	MD	700	1/54-12/78	40.5	12.0	Silt Loam	140	280
13	SC	1726	1/54-12/78	47.1	16.7	Sandy Loam	105	295
14	IA	205	1/39-12/63	31.7	9.8	Silt Loam	152	288
15	MS	1707	1/54-12/78	50.4	17.1	Silt Loam	152	305
16	IN	6506	1/54-12/78	40.8	11.1	Silt Loam	140	283
17	SC	1726	1/54-12/78	47.1	16.7	Sandy Loam	152	319
18	IA	1411	1/54-12/78	50.4	17.9	Silt Loam	105	263
19	AL	5140	1/49-12/73	52.1	17.7	Sandy Loam	105	263

1" = 2.54 cm

their respective areas summed. In most cases a single soil type was prevalent. The soil having the largest areal coverage was chosen. These assumed soil textural classifications for each site are also listed in Table 2. These textures were used to determine field capacity (θ_{fc}) and wilting point (θ_{wp}) soil water contents and bulk density (ρ_s) inputs for PRZM.

Crop information was also determined by consulting SCS soil surveys, the PRZM Model User's Guide (Carsel et al., 1984), and Agricultural Handbook #283 (U.S.D.A. 1972b). Planting and harvest dates from Handbook #283 for the major crop at each site are also listed in Table 2. The plant rooting depth was taken as the average of the range reported by Carsel et al., (1984).

Frequency Distribution Production

After choosing the appropriate range of input parameter values, and the representative sites, the final step was to make the simulation runs required to produce the frequency distributions. One hundred runs (5 R values x 5 decay rates x 4 curve numbers) of 25 year duration were made for each site, resulting in 2500 simulation years per site. The frequency distributions developed from the simulation output are a compilation of nineteen multiples of these runs, or 47500 simulation years. The simulations were made on an HP-1000 microcomputer.

Assessment of Leaching Potential

The procedure for assessing significant pesticide leaching is shown schematically in Figure 5. The first step is to select a crop and site targeted for use of the pesticide. Once this has been done, data can be gathered to evaluate the three key parameters, essential to the methodology; the pesticide decay rate, the retardation factor for the pesticide/soil combination and the runoff curve number for the soil/crop management scenario. Each site number (1 through 19) corresponds to a matrix in the LEACH report (A1 through A19). These matrices are used to determine if significant leaching occurs and to locate the cumulative frequency curves which indicate severity.

The user inputs required to perform an assessment are few, but must be determined from a variety of sources. Much of the information can be found in tables located in the appendices of the LEACH report or in referenced documents. To apply the methodology the user must choose representative values for:

- site characteristics
  - hydrologic soil group
  - soil organic carbon and bulk density

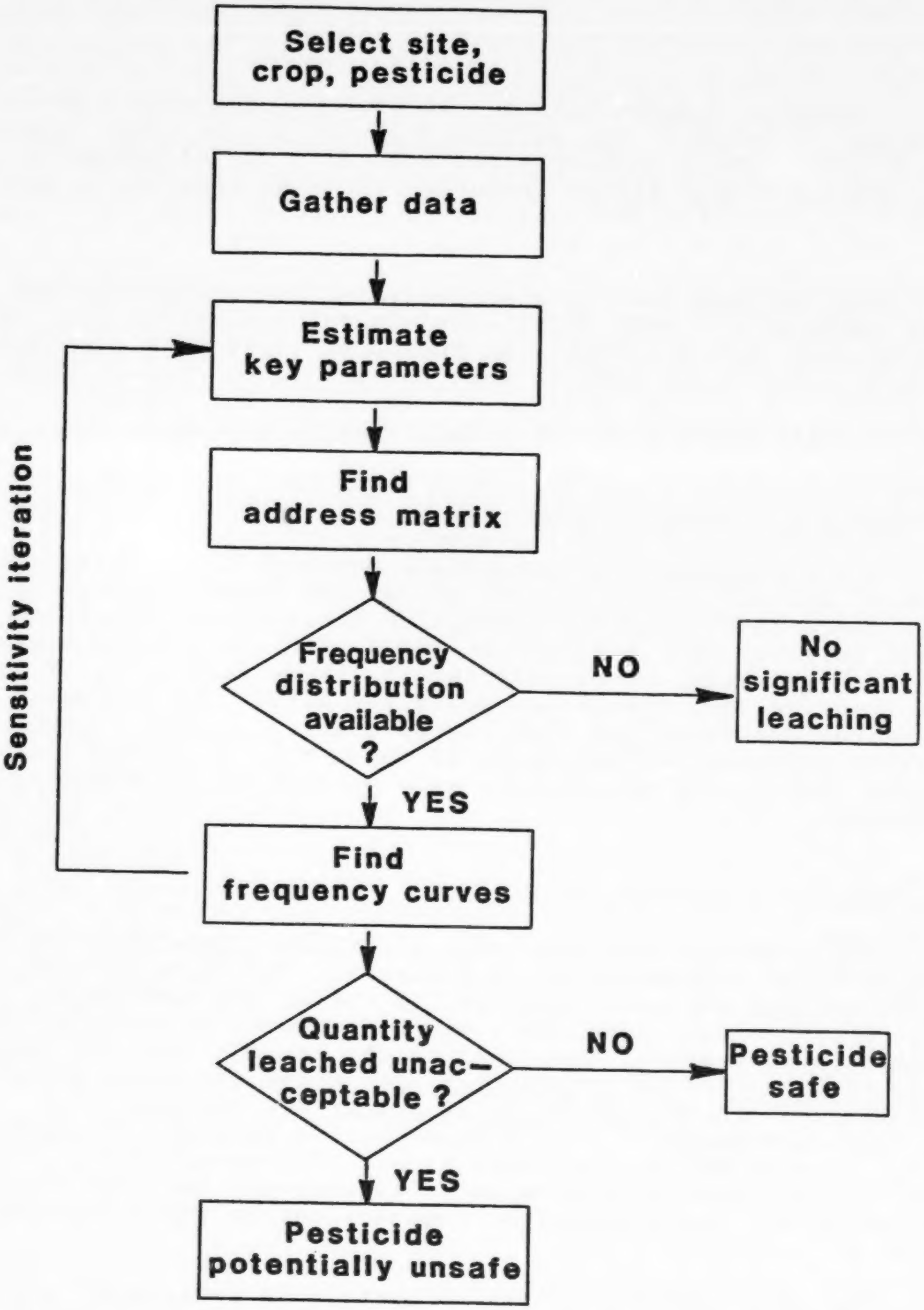
Select site, crop, pesticide
Gather data
Estimate key parameters
Find address matrix
Frequency distribution available ?
NO
No significant leaching
YES
Find frequency curves
Sensitivity iteration
Quantity leached unacceptable ?
NO
Pesticide safe
YES
Pesticide potentially unsafe

Figure 5. Schematic for Pesticide Leaching Evaluation.

- crop type
- pesticide properties
  - organic carbon partition coefficient, K_{oc} (or K_{ow})
  - decay rate, k_s, and
- management practices (used to determine the SCS curve number)

A chart showing how the above information is used to estimate R, k_s and CN appears in Figure 6.

Site Selection

The site selection procedure requires only knowledge of geographical location and crop type of concern. Four maps are provided showing representative sites for wheat, corn, soybeans, and cotton. It is necessary to use the map that corresponds to the crop of interest. The user simply selects the representative site into which his location of interest falls.

Although representative sites cover the major crop growing areas, these crops are often grown outside the regions shown. If the area of interest does lie outside of the site boundaries, the user should examine the precipitation and soils characteristics in the area of interest and choose a site with the same crop and similar characteristics. The characteristics that should conform are average annual precipitation, hydrologic soil group, soil texture (sand/slit/clay), soil organic carbon, and suitability of the soil for growing the crop of interest.

Parameter Estimation

Retardation Factor, R

The retardation factor requires determination of the chemical partition coefficient (K_d), the soil bulk density (ρ_s), and the average soil water content at field capacity (θ_{fc}). The equation is

$$R = 1 + \frac{K_d \rho_s}{\theta_{fc}} \tag{3}$$

where R is dimensionless, K_d has units of cc/g-soil, ρ_s has units of g-soil/cc and θ_{fc} has units of cc/cc. K_d is given by

$$K_d = K_{oc} (OC)/100 \tag{4}$$

where K_{oc} is the organic carbon partition coefficient having units of cc/g-organic carbon and OC is the percent soil organic carbon. If K_{ow} (octanol-water partition coefficient) or S

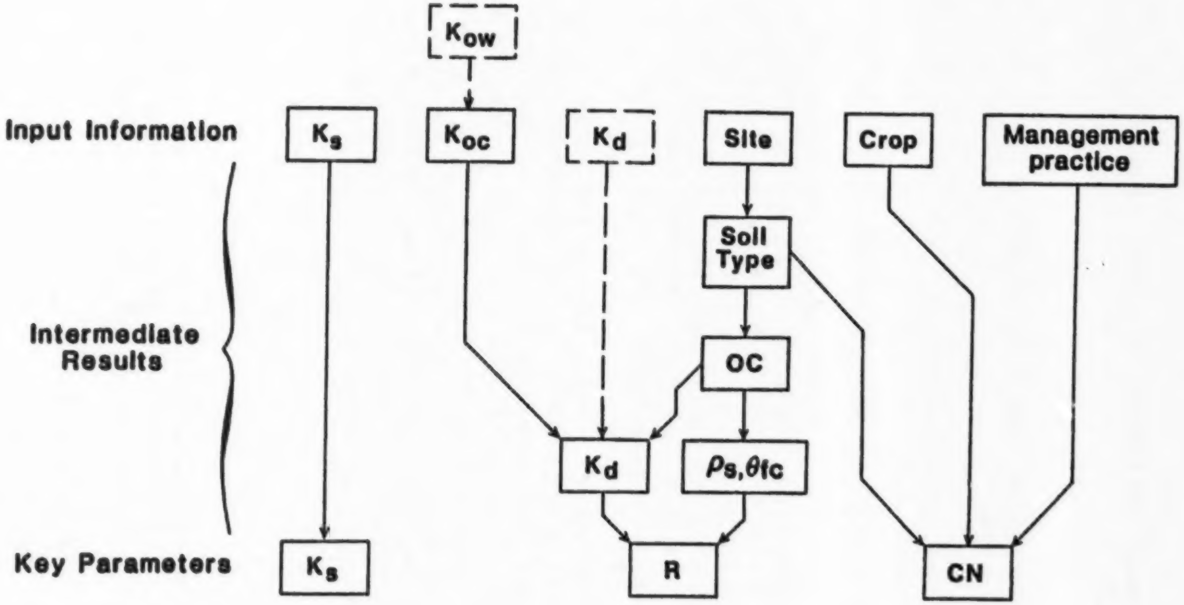
Input Information
Intermediate Results
Key Parameters
K$_s$
K$_{ow}$
K$_{oc}$
K$_d$
Site
Crop
Management practice
Soil Type
OC
K$_d$
ρ_s, θ_{fc}
K$_s$
R
CN

Figure 6. Schematic of Information Flow to Calculate Methodology Key Parameters.

(solubility) is reported instead of K_{oc}, several empirical funtions can be used to convert to K_{oc} (Lyman, et al., 1982). An excellent source of solubility data for herbicides is available (Weed Science Society of America, 1983) if the compound's K_{ow} or K_{oc} cannot be found in the literature. 16.

No compilations of soil organic carbon on a national scale have been found in the literature. In the absence of this specific data, soil nitrogen content can be used as a surrogate. The organic carbon to nitrogen ratio in the soil is approximately 11 to 1 (Brady, 1974). Nitrogen content of surface soils in the United States can be estimated from a map in the handbook.

Once the soil type is known, the textural classification provides sufficient information to estimate bulk density (ρ_s) and the field capacity water content (θ_{fc}). If the user has access to profile information for the soil of interest, it may be desireable to either use weighted mean values of OC, ρ_s and θ_{fc} for the profile, or introduce this variability into sensitivity analysis of the retardation factor.

Decay Rate, k_s

Values of k_s for many agricultural chemicals are provided in appendices in LEACH. Extreme care should be taken in using the decay rates reported. There are two important potential problems. First, these rates have been measured under a wide range of environmental conditions, (e.g., varying soils, soil pH, soil organic matter, soil moisture, soil temperature, etc.) which can have dramatic effects on reported values. Secondly, in this methodology, the decay rate is applied to both dissolved and adsorbed chemical. The user should be satisfied that the conditions under which the decay rates were measured are relevant to his or her situation.

For compounds not covered in LEACH, registration documents for the specific pesticide available through the U.S. EPA or the chemical manufacturer may contain appropriate values. For use in this methodology, the units for k_s must be day^{-1}.

Curve Number, CN

To incorporate both soil characteristics and land management practices into the system, the Soil Conservation Service curve number method is used. To estimate CN soil infiltration characteristics, watershed hydrologic conditions, and conservation management practices should be known.

Each soil belongs to a hydrologic soil group, designated A, B, C, or D. These index letters define the soil's infiltration capacity. Most soils of the U.S. have been classified by the U.S.D.A. (1972a) using this system.

Cropping practices as well as soil type affect the soil's ability to transmit water. The hydrologic condition reflects the amount of dense vegetation in each rotation. Poor hydrologic conditions result from repeated rotations of the same crop in the same field. Good hydrologic conditions result from incorporation of legumes or grasses in rotations to improve the infiltration characteristics of the soil.

Land use management information also affects curve number selection. Variation in curve numbers reflects runoff potential associated with straight row, contoured, or contoured and terraced lands.

Given the above information a runoff curve number can be selected from tables in LEACH. The runoff curve number technique is well known and is not presented again here.

Selection of Frequency Distributions

Once values of R, k_s, and CN are known, CDFs (cumulative distribution functions) which correspond to these values are found by using matrices found in the handbook. The matrices are ordered by representative site numbers 1 through 19. Each two-dimensional matrix features retardation coefficient (R) and decay rate (k_s) along the axes. The body of the matrix contains figure numbers which direct the user to the appropriate plot. Blank locations in the matrix are indicative of parameter combinations that result in no significant leaching.

Each plot contains leaching frequency distributions for a single value of R, a single value of k_s and four values of CN. The horizontal axis is the percentage of the annual pesticide application leached per year and the vertical axis represents the percentage of time that the specified amount of pesticide leached will be exceeded. The following is a step-by-step example of how the plots are located and interpreted.

1. Assume that we are interested in a site located in northeastern Colorado planted in corn. Figure 2 shows that representative site #7 corresponds to this location.

2. We calculate input parameter values for R, k_s, and CN (e.g. $R=1$, $k_s=.010$, CN= 80-85). Note that computed values of R, or k_s may not match exactly those found in the matrix. In this case a value appearing in the matrix closest to that calculated is used.

3. Figure 7 contains the example matrix used in locating the appropriate CDF.

SITE NO. 7
CROP: CORN
CN: 78-85

	ks				
R	.001	.005	.010	.050	.100
1	A7-1	A7-2	A7-3	A7-4	A7-5
3	A7-6	A7-7	A7-8	A7-9	A7-10
5	A7-11	A7-12	A7-13		
20	A7-14	A7-15			
50					

Figure 7. Example Address Matrix.

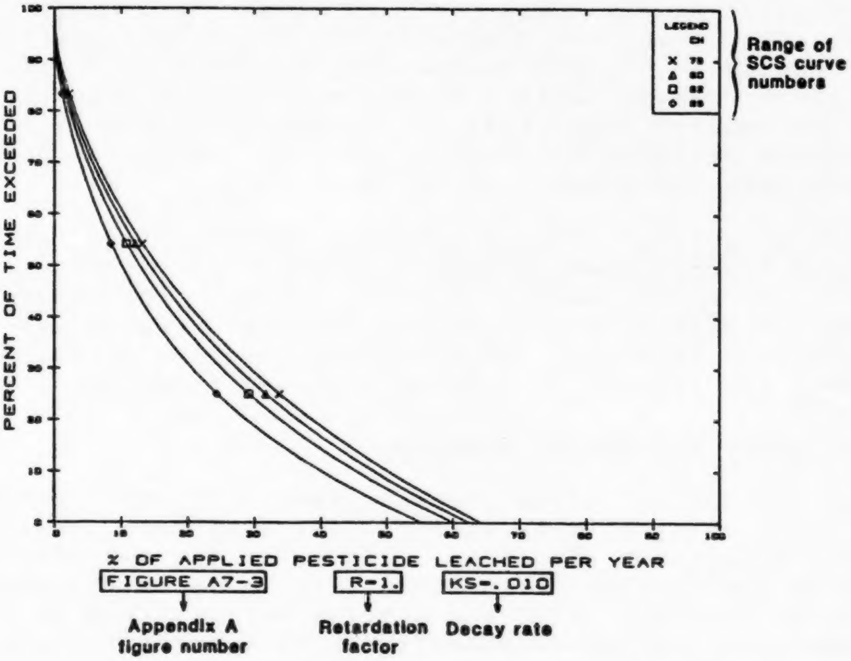

Figure 8. Example Frequency Curve Plot.

4. According to the key parameter values from step 2, the corresponding frequency distributions can be found in Figure A7-3. Normally, a range of values for R and k_s would necessitate looking at several plots.

5. Figure 8 contains the example plot A7-3 with explanations of its format. The frequency curves are read as follows. For R=1, k_s=.010, and CN=85; there is, for example, a 10% chance that more than 40% of the applied pesticide will leach per year. For the same conditions, there is a 50% chance that more than 10% of the applied pesticide will leach per year.

Since the percentage leached is independent of application rate, the leached load can be expressed as the applied load times the percent leached. Thus, if 5 Kg/ha were applied, there is a 50% chance that more than 0.5 K_g/ha will leach past the root zone in any given year.

The family of curves shown on each plot encompasses the range of curve numbers used for each site. This range of curve numbers reflects the range of soil types and management practices reported in the SCS Engineering Handbook (U.S.D.A. 1972b). In the above example, site 7 displays characteristic curve number values ranging from 78-85. This was deduced from the site descriptions defining the soil as part of Hydrologic Soil Group "C" and planted in a row crop, corn.

Just as a single figure contains distributions for a range of SCS curve numbers values, a group of figures contains distributions for a range of R or k_s values. For example, plots A7-1 through A7-4 show the results of varying k_s values from .001 to .050. Likewise, distributions found on plots A7-6 and A7-11 show the results of varying R values from 3 to 5.

Interpretation and Use of Results

The methodology gives the user very specific information about the leaching of a pesticide under a certain climatic/soil/crop/management situation. First, if the retardation/decay rate combination for a given site falls in the blank lower right hand corner of the address matrix, then no significant leaching (<0.05% of annually applied quantity) was observed in the 25 year simulation period. If sensitivity iterations give combinations which all fall into this region, then it is a good assumption that no significant leaching would occur from the use of the compound. It must be remembered, however that pesticide toxicity plays a major role in determining if the presence of a given level of pesticide in water or soil is safe or unsafe. The 0.05% level was somewhat arbitarily chosen but probably represents a level that is safe given the toxicity of most currently available pesticides.

For borederline cases in which sensitivity analyses indicate some significant leaching situations or for cases in which large amounts of pesticide would appear to be leaching, subsequent "post screening" analyses are warranted. The CDFs will be useful in such cases.

The CDFs give an estimate of the probability that a certain annual pesticide loading rate will result below the crop rooting depth. The next logical step in determining if the use of the pesticide constitutes a substantial risk to the environment is to develop concentrations in ground water.

To calculate these concentrations the mass of pesticide leached must be mixed in some volume of water. This can be easily done but there are several cautions worth mentioning in this regard, underscored in the following discussion.

A simplified scenario of pesticide loading to ground water is shown in Figure 9. The source area discharges leachate directly to the aquifer in this case. Notice that the bottom of the root zone does not necessarily coincide with the ground water table. The CDFs which result from application of this methodology give the mass loadings exiting the root zone a certain percentage of the time for a unit portion of the source area. Unless the ground water table is very close to the bottom of the root zone, the quantity of leachate reaching ground water may be different from that exiting the root zone due to decay of the contaminant. Of course a "worst case" scenario would assume that no decay occurs; hence, the bottom of the root zone loadings would equal ground water loadings under "steady-state" conditions. The chemical will then be mixed in ground water to a certain depth (the mixing depth). By knowing the area of the source (assuming that this area and the contact area of ground water by leachate are equivalent) and the mixing depth, a volume of contaminated ground water can be determined immediately beneath the source and a concentration can be obtained. An inherent assumption in this approach is that the recharge volume from the source area is negligible compared to the mixing volume.

Contaminated water will be advected in the direction of ground water flow. Normally, the mixing depth becomes greater in this direction because of diffusion and dispersion of the chemical. The plume also will spread horizontally. Due to this dispersion, decay of chemical, and possible additional recharge, the concentrations will decrease as the chemical moves downgradient in the aquifer. Thus concentrations depend crucially at where and when one looks in the aquifer.

Further complications arise when confining layers occur

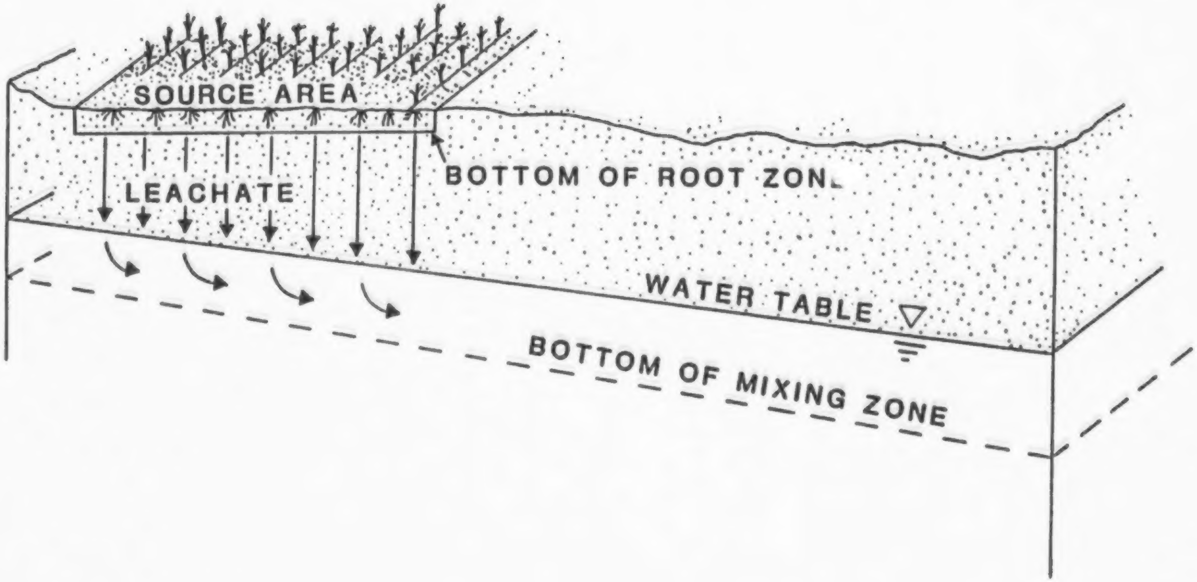
SOURCE AREA
LEACHATE
BOTTOM OF ROOT ZON
WATER TABLE
BOTTOM OF MIXING ZONE

Figure 9. Schematic of agricultural chemical contamination of ground water under a scenario of direct recharge.

between the root zone and the ground water aquifer.

The type of analysis described above can only be done by using specific localized information, such as depth to ground water and transport and fate characteristics in the aquifer. Concentrations can be calculated at the screening level by using simplified techniques e.g., Donigian, et al., (1983); Brown (1983), or at a higher level by using numerical ground water simulation models of which there are many. In either case, the CDF is used as a "loading function", and the ground water model as a transfer function to generate frequency distributions of concentrations at various times and locations in the aquifer.

Notice also that in the preceeding discussion it was stated that the CDFs give an estimate of pesticide leaching probability. Only a single 25 year historical sample of meteorological data was used to generate these estimates. To get a better estimate, many 25 year sequences might be run. This would result in a "distribution about the CDFs" from which confidence limits could be estimated. This has not been done because of the obvious time and expense required. Another way of "improving" this estimate would be to fit probability density functions (PDFs) to each sample density function. This would result in "tails" to the right on each CDF instead of the oblique angles at which some curves intersect the x-axis on the frequency plots. Because no one PDF could be found to fit all the samples and because of the small sample size (25 points) this was not done. Tail probabilities based on such a small sample might be misleading. Therefore the samples were simply fit with 4th-order polynomials and the resulting curves appear in the report. The percent of time that annual pesticide leaching exceeds a given level should not be strictly interpreted as a probability.

Application Assumptions and Limitations

The leaching frequency distributions in the LEACH handbook were created by applying PRZM under 19 idealized scenarios. Various simplifying assumptions were made which are important to the interpretation of methodology results and may also affect applicability in certain situations. Major assumptions and limitations of the methodology are listed below.

Assumptions

Soils

- A uniform soil profile was assumed from the surface to the crop rooting depth.

Crops

- Representative crop rooting depths were chosen for each crop type and were the same for each site in which the crop is grown. The average values reported by Carsel, et al., (1984) were used. These values were for corn, soybeans, wheat and cotton; 90, 45, 22 and 60 cm, respectively.

Management

- One pesticide application per year was made during each year of the simulations. The application was made just prior to the time of planting. There are many cropping patterns and pesticide application choices available. For instance, insect control in cotton may require foliar application several times during a growing season while weed control may be accomplished with one surface application prior to planting. The full range of possible combinations was too extensive to treat on a national scale. The majority of pesticides which would cause significant leaching problems are herbidices or fungicides applied prior to crop emergence, so an appropriate application date for each site was chosen. To best simulate actual application conditions, the pesticide was applied at least two days following a rain event as well as at least two days prior to the next rainfall. This emulates a drying period following events sufficient to allow farm equipment use and a post application period reflecting the farmer's probable use of weather forecasts.

To test the validity of assuming a single pesticide application scenario, an analysis was performed on the effect of making a single annual application as opposed to 10 applications over a 100 day period. The results are displayed in figure 10. It can be seen that the greatest discrepancy in pesticide remaining at the end of the 100 day period occurs in the middle of the range of

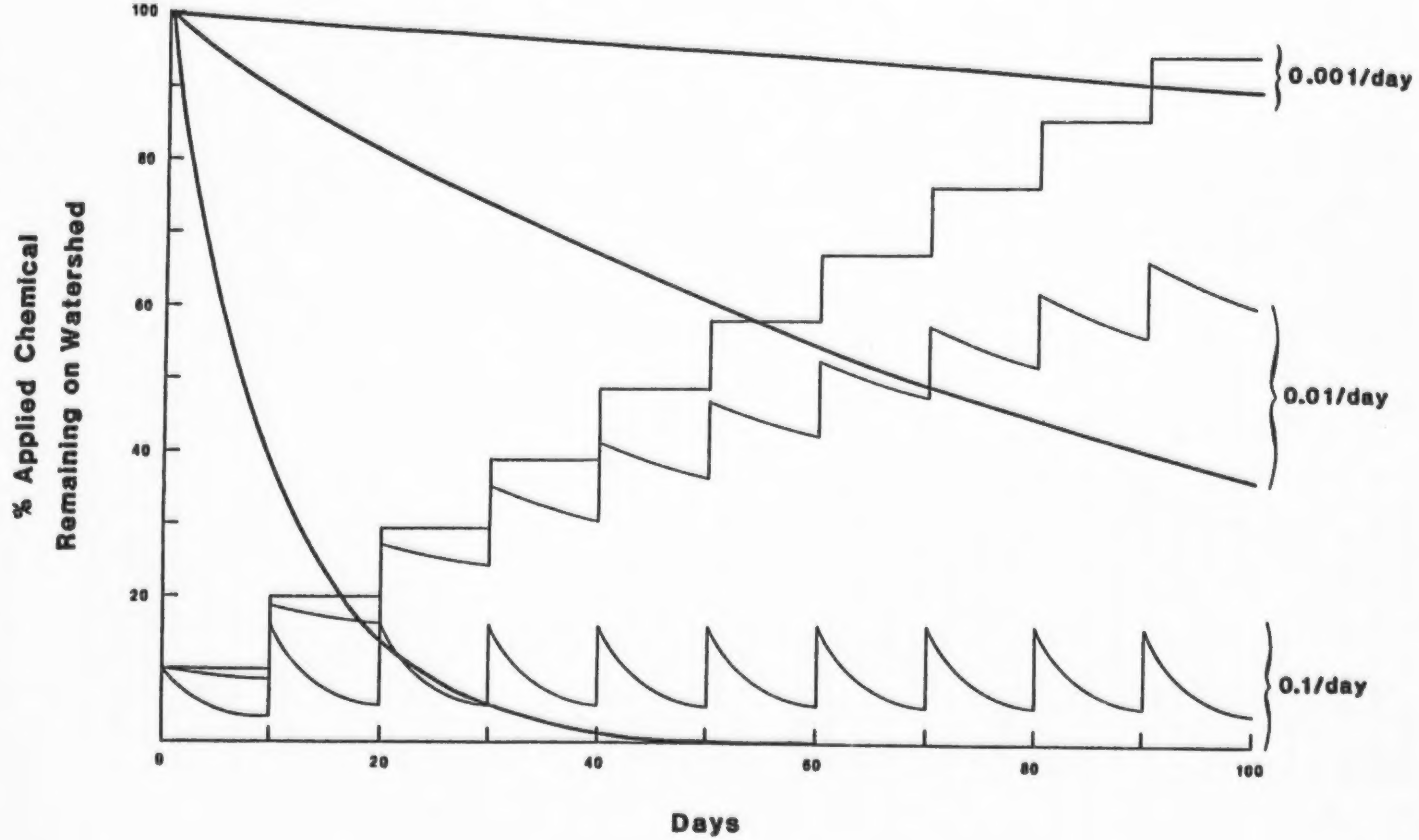

Figure 10. Comparison of Effects of Single and Multiple Pesticide Applications for Various Decay Rates.

probable decay rates. By applying pesticide continuously over this period, there will always be more chemical in the soil profile at the end of the 100 day period than if it is applied only once. In the cases where the decay rate is either very large or very small, little difference will be seen at the end of the 100 day period if application occurs once or at several times during the growing season. However in all three cases there is a discrepancy between pesticide available for leaching at the beginning of the 100 day period under the two different application scenarios. This effect disappears most quickly at high decay rates. Therefore, if the single application assumption is made and multiple applications are in fact used, at low decay rates actual pesticide leaching would be over-estimated while at high decay rates, leaching would be underestimated.

The assumption of the application of the pesticide each year has no impact on the frequency distributions for short lived chemicals. However, for chemicals that carryover from year-to-year, leaching would be overestimated. A single cutoff value of half-life (or decay rate) cannot be given because it changes with each representative site and value of K_d.

- One crop per year was simulated instead of using a rotation scenario and the same crop was simulated each year at each representative site.
- Pesticide was surface applied rather than incorporated into the soil in the model. Pesticide incorporation would reduce those surface losses such as volatilization photolysis, and runoff. Therefore, lower overall model decay rates would be associated with subsurface incorporation.

Limitations

The greatest limitation of this methodology concerns the use of the characteristics of a single soil and single meteorological station to represent large areas. The handbook was devised as a screening tool for evaluating the potential for pesticide leaching to ground water. If site-specific characteristics differ radically from those of the representative site, further investigation may be necessary.

Another limitation is that available chemical data is both incomplete and imprecise. Reported values for K_{oc} and K_{ow} vary widely for the same or similar compounds. In many cases the coefficient of variation is over 100% of the mean value reported. The user may find that by taking the full range of reported values in estimating the three key parameters the entire range of retardation factors or decay rates used in the methodology will be spanned. In this case, the user should strive to narrow the uncertainty in his input data and re-evaluate.

Summary

A screening methodology was devised to detect large areas (on a national scale) in which certain combinations of pesticide/soil and climatic parameters might combine to favor contamination of ground water due to pesticide leaching (Dean, et al., 1984). Four major crops were investigated; corn, soybeans, wheat and cotton. Factors important to pesticide leaching such as precipitation, soil infiltration capacity, soil organic carbon, etc. were used to delineate regions thought to differ in their pesticide transfort potential. In each of these areas, the Pesticide Root Zone Model (PRZM, Carsel, et al., 1984) was used to simulate a 25 year time series of pesticide leachings past the crop rooting depth. These time series were expressed as annual values of percent of applied chemical leached and analyzed by constructing frequency histograms. These histograms were organized by region and indexed using matrices. By determining three key parameters - retardation factor, pesticide degradation rate and runoff curve number, the user can determine if significant quantities of pesticide will leach under a particular pesticide/site/crop/management scenario.

Acknowledgement

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VEGETATION MANAGEMENT: A SOLUTION TO SHALLOW GROUND WATER CONTAMINATION IN NORTH-CENTRAL MONTANA

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Shallow ground water quality over portions of the Northern Great Plains has degraded considerably over the past 20 to 40 years. Shallow wells in the affected areas have shown a range in TDS from 20,000 to 50,000 mg/l. Although the prevalent geologic and climatic conditions are significant, the degradation is attributed to, for the most part, the cultural farming practices of the region. The use of a strict, alternate crop/fallow management practice results in the inefficient use of annual precipitation and allows for the downward migration of subsurface salts into shallow ground water systems. In addition, under the right geologic conditions, the salts may resurface and destroy productive cropland in the form of saline seep. It has been estimated that 280,000 (113,300 ha) acres of cropland have beeen lost to this process in Montana.

In north-central Montana, a group of local landowners, in cooperation with state and federal officials, have formed a technical team to address the problem of saline seep and the degradation of shallow ground water quality. An intensive management plan is developed that reclaims salinized cropland and prevents further degradation of ground water quality. These management approaches require the use of vegetative systems that efficiently use soil moisture and natural precipitation in a manner that prevents percolation of water into the shallow ground water system. The use of a deep-rooted perennial (i.e. alfalfa) or change to a flexible cropping program, has led to a significant decrease in the size of saline seeps in as little as 3 to 5 years. In addition, the knowledge gained has led to potential use of vegetative management as a reclamation tool for other water quality problems such as acid mine drainage.

Introduction

There exists, in north-central Montana, a serious resource problem that has drawn considerable attention over the last 10 to 15 years. This resource problem is manifest as salinized ground within formerly productive cropland. Saline seep is defined as recently developed, saline soils in nonirrigated areas that are intermittantly to continuously wet, often with white salt crusts, and result in the decrease or elimination of crop germination and growth. At present it is estimated that there is a total of 280,000 acres (113,300 ha) of cropland in Montana that are affected by saline seep. The magnitude of this problem becomes more dramatic when it is noted that this figure is increasing approximately 10% per year.

The initial attention to saline seep focused on the decrease in crop yields and eventual total loss of affected cropland. However, the most severe consequence of this resource problem is the degradation of local surface water and shallow ground water quality. Only recently has the degraded water quality become a more important concern than the lost cropland, since the salinized land can be more readily reclaimed. Wells and ponds close to salinized areas have shown a 10 to 20 fold increase in total dissolved solids (TDS) in the water over the last 10 to 15 years. This deterioration in quality renders the water useless for either human or livestock consumption and necessitates the search for a new "fresh" water supply. Although it is difficult to quantify the number of domestic and stock water sources lost to saline seep, in many parts of north-central Montana the practice of drilling a new water well every decade has become culturally ingrained. If saline seep is allowed to grow unchecked, the potential for a region-wide deterioration of ground water resources looms on the horizon.

The Triangle Conservation District (TCD) was formed to address the problem of saline seep and the degradation of shallow ground water systems. Working on a farm-by-farm basis, an intensive management plan is developed to reclaim salinized cropland and shallow ground water sources through a change in vegetative land use. The objective of this paper is to describe the condition and factors for saline seep development and detail the use of vegetative management as a reclamation tool.

Saline Seep Development

The formation and development of saline seep under variable conditions has been thoroughly discussed in previous works (Dodge et al 1984; Brown et al 1982; Miller et al 1981; Brown and Krall 1979; Vander Pluym 1978; Halvorsen and Black 1974; Bahls and Miller 1973). Due to the profusion of available literature, we would like to limit our discussion to a review of saline seep development and the hydrogeologic conditions as they exist in the Triangle area of central Montana. (See Figure One).

Saline seep formation is actually the result of an interaction of climatic and hydrogeologic conditions with local cultural practices. The climate of north-central Montana is arid to semi-arid with widely variable precipitation [10 to 18 inches (25 to 45 cm)], low humidity, warm summers and cold winters. Winter low pressure centers sweep southwesterly out of Canada bringing strong winds that result in generally limited snow cover [less than 2 to 3 inches (5 to 8 cm)] and extensive drifting. The variable precipitation patterns, low humidity and high evapo-transpiration rates during the growing season have made moisture one of the principle growth limiting factors of northern Montana's agricultural industry.

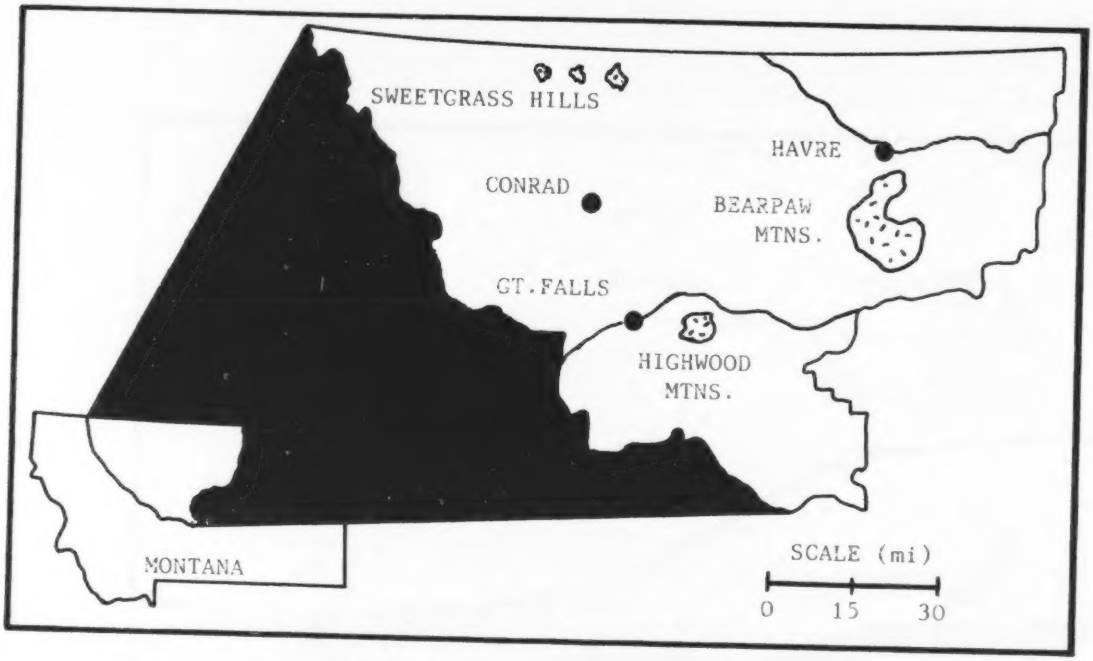

Figure 1. Location of the "Golden Triangle" in North-Central Montana

The largest land use change in the Northern Great Plains since the 1930's has been from native range to a rigid crop-fallow rotational system (Dodge et al 1984). A strict crop-fallow rotation has a year of small grain production followed by a year of fallow, or no crop growth. This management strategy was encouraged as a method of slowing erosion and conserving soil moisture. In reality, the rigid crop-fallow cropping system does not efficiently use the annual precipitation. In north-central Montana, native range uses water approximately 14 months out of a 24 month period while crop-fallow allows crop water use only 3 to 6 months out of this same 2 year cycle. During a fallow year, most soils are capable of storing from 4 to 10 inches (10 to 25 cm) of moisture in a four foot rooting depth. Thus, any precipitation in excess of the soil's moisture holding capacity is lost to deep percolation. Additionally, other cultural features (shelterbelts, windbreaks, roadways and snow fences) may cause excess snow accumulation or restrict drainage and thereby accentuate the inefficiency of the system.

Normally the water table under fallow ground shows a rapid rise [up to 10 feet (3 m)] during spring melting that is followed by a gradual decline over the remainder of the growing season. It has been noted (Miller et al 1981) that the water table lows do not reach the previous year's low. This seems to imply a water table buildup due to deep percolation of unused soil moisture. As the water migrates vertically through the soil, it dissolves soluble salts present within the profile. Upon reaching a less permeable layer (typically shale) the ground water begins to flow laterally down gradient. This salt-laden fluid can resurface on encountering a change to heavier-textured soil, decreased gradient, or physical barrier. With ground water flow restricted, a water table buildup occurs. Once the water table is within 3 feet (0.9 m) of the surface, capillary action and evaporation account for the soluble salts being precipitated out at or near the surface.

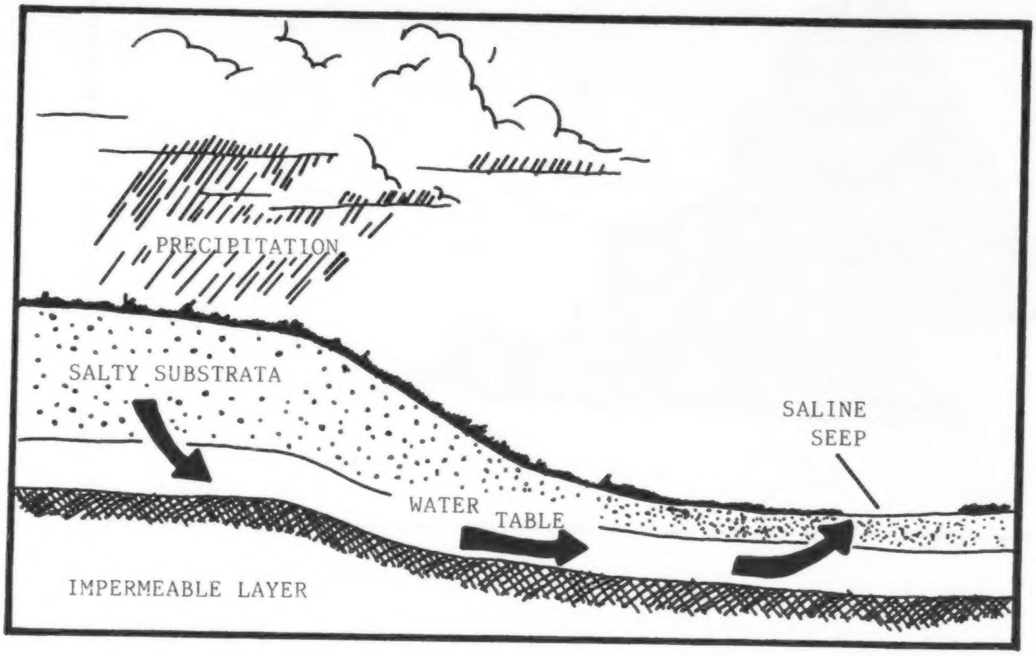

Figure 2. Typical process of formation of saline seep in North-Central Montana.

North-central Montana's geologic history has been characterized by long periods of acquiescence which allowed for the deposition of several thousand feet of mainly marine sediments. This was the case throughout the Paleozoic (225 m.y. to 600 m.y.) and much of the Mesozoic (70 m.y. to 225 m.y.) eras. Mid-Cretaceous time (83 m.y. to 105 m.y.) in this area was marked by the widespread deposition of over 2,000 feet (610 m) of marine shale (Colorado Group) on a relatively stable, epicontinental sea. As such, the Colorado shale is a black marine shale with numerous concretions and thin bentonite beds in its upper portions. The Colorado epoch was brought to a close by uplift of the present Rocky Mountains on the western edge of central Montana. This period of crustal instability produced multiple transgressions and regressions of the Cretaceous sea and imparted a cyclic nature to the succeeding Montana Group (Telegraph Creek, Eagle Sandstone, Claggett Shale, Judith River, Bearpaw Shale, Fox Hills and Hell Creek formations). Each of the transgressive facies (Claggett and Bearpaw) deposited marine shales that vary only slightly from the Colorado Group. A regional north-east tilt of less than 3° accounted for the erosion of Early Tertiary (50 m.y. to 65 m.y.) sediments and the exposure of the Cretaceous marine shales. Two or more glacial ice advances during the Pleistocene (15,000 to 2 m.y.) left behind a thin mantle [3 to 80 feet (1 to 24 m)] of poorly sorted, unconsolidated deposits. The glacial till greatly altered the pre-existing landscape by filling previous lows and forming a gentle, rolling terrain.

The main portion of north-central Montana is underlain by three main complexes: (1) Colorado Shale, (2) Claggett Shale-Judith River Formation, and (3) Bearpaw Shale.

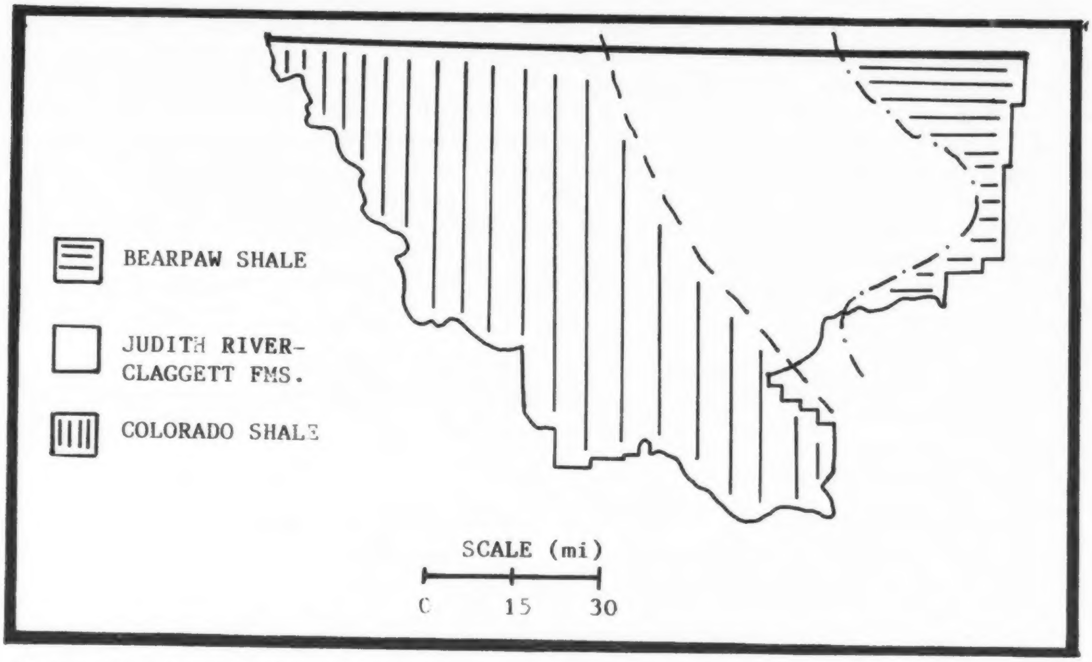

Figure 3. Distribution of upper Cretaceous sediments in North-Central Montana

As identified by Miller et al (1981), the importance of the area geology is in that:

1. The bedrock units influence the type of material and amount of salt that are incorporated in the overlying glacial drift and associated ground water.
2. The fresh shale and thin bentonite beds create an impermeable layer thereby prohibiting any significant ground water movement through these units.
3. The weathered shale zone provides a laterally continuous permeable zone allowing water to migrate down gradient.
4. The poor drainage associated with glacial terrains allows water to pond for extended periods of time.
5. The extensive vertical joints and fractures of the glacial till allow water to percolate through the drift.
6. The drift contains an abundant supply of water soluble salts throughout the region.

The shallow ground water quality in areas of high saline seep density is very poor. In general, the water quality corresponds to the underlying geology - a TDS concentration from 20,000 to 55,000 mg/l in Colorado Shale and Bearpaw Shale with a range of 10,000 to 25,000 mg/l in the Claggett Shale-Judith River unit (Dodge et al 1984). The major constituents present are soluble calcium, magnesium and sodium sulfates with lesser amounts of nitrates, chlorides and bicarbonates. In addition, in a study by Miller et al 1978, 40% of the water samples collected showed increased levels of selenium (greater than the 0.01 mg/l limit by the U.S. Public Health Service). Table 1 shows an averaged water quality analysis from an area underlain by Colorado Shale and from an area underlain by Judith River-Claggett formations. The increased salt load of ground water in the region has very serious implications. Not only have domestic wells had to be abandoned, but livestock and fish kills have been attributed to salinized surface water supplies. In one extreme case, a flowing saline seep salinized 2.5 miles (4 Km) of drainageway and had a salinity level of 78,000 TDS immediately before entering a lake. To ensure an adequate and continued "fresh" water supply in north-central Montana, it is necessary to re-isolate these soluble salts before further degradation of water resources occurs.

Table 1. Averaged analysis of ground water from areas underlain by Colorado Shale and Judith River-Claggett formations (from Miller et al 1981).

	Colorado Shale Unit	Judith River-Claggett Eagle Unit
	Mean Concentrations (mg L^{-1})	
Ca	275	120
Mg	866	71
Na	1317	670
K	17.7	6.1
Fe	0.3	0.3
SiO_2	12	12
HCO_3	534	722
Cl	141	177
SO_4	6041	1143
NO_3	57	6
Se	0.308	0.028
Sr	4.6	2.0
TDS	9262	2928
pH	7.62	7.89

Vegetative Reclamation

The practice of summer fallowing to control weeds and conserve moisture became popular following the drought years of 1917-20. The accompanying strip-cropping to prevent wind erosion became universal in the 1930's with government incentive payments. The disadvantages of summer fallow associated with saline seep development are the inefficient soil water storage capabilities and annual precipitation use. In many cases more precipitation is received in a 2-year period than can be stored in the 14-21 months of fallow or utilized by crops in the remaining 3-10 months. Approximately .75-1.25 inches (2-3 cm) of water per year moves downward under fallow conditions (Ferguson et al 1978). The amount lost can be much more on above average rainfall years or in light textured soils with low water holding capacities. This process has been going on since the late 1940's due to the advent of large scale farm equipment which controls virtually all moisture robbing vegetation in the fallow period.

The saline seep can be divided into two areas defined as the discharge and recharge. The discharge area is the wet, salinized soil profile which is actually the symptom, not the problem. The recharge or upslope area is where the seep water originates as unused precipitation. Before the seep itself can be reclaimed, a more intensive crop water use program must be initiated on the recharge area. The drilling of shallow ground water monitoring wells defines the recharge area and determines the soil profile, depth to bedrock and amount of stored moisture. From this information, a site specific intensive cropping system is designed, keeping in mind the climate and management of the individual land operator.

The recharge area treatment must be as intensive, i.e. plant growth as long and often, as is economically feasible. Alfalfa (Medicago sativa) has proven to be the most consistent crop recommended because of its many attributes.

Alfalfa benefits, both long and short term, include:

1. Ability to root deeply to utilize stored soil moisture below the normal or small grain root zone, and to utilize moisture from the capillary fringe of the water table.
2. Long growing season, whereby it commences growth early in the spring and continues to use water into the fall or about 7 months. The precipitation-use efficiency is 95-100%.
3. Economic yields comparable to cereal grain production if managed properly, i.e. adequate seedbed, fertility, rhizobia bacteria inoculation, harvesting, and length of stand.
4. Improved soil tilth, and increased organic matter and nitrate levels (important for subsequent crops).

Alfalfa will dry out the deep subsoil to provide a "dry reservoir of soil" for storage of unused precipitation for cropping subsequent systems. The alfalfa need not be a permanent stand, but is usually left in for 5-6 years depending on the water table levels and the quality of the stand. The cultivars adaptable to this area vary considerably in rooting depth and soil water depletion. Brown et al (1983) determined that after 5 years Beaver alfalfa could root to 24 feet (7.3 m) and used 41 inches (104 cm) of water (in addition to the annual precipitation), while Kane rooted only 16 feet (4.9 m) and used 21 inches (53 cm) of water. In the same studies, other perennial legumes and grasses were found to vary significantly, but used less water than alfalfa and more than small grain production (Table 2).

Table 2. Rooting depth and net soil water depletion for perennial vegetation after 5 years compared to annual crops, all grown near Fort Benton, MT. (Brown, et al 1983).

Crop	Rooting Depth	Net Soil Water Depletion
Perennial	Feet	Inches
Beaver alfalfa	24	41
Ladak 65 alfalfa	22	26
Kane alfalfa	16	21
Intermediate wheatgrass	15	29
Crested wheatgrass	13	16
Annuals		
Safflower	7	10
Sunflower	6	7
Winter wheat	6	7
Barley	5	6

Intensive cropping systems are implemented on the recharge area when 1) the alfalfa is removed, 2) alfalfa is not an economically viable alternative or 3) the soil profile is too shallow or too permeable to support a healthy stand. The two cropping systems recommended by TCD are annual and flexible. Annual cropping or one crop each year is appropriate under very limited conditions, therefore not widely used. If the soil profile is shallow, less than 5 feet (1.5 m), or very permeable, the water holding capacity is limited, therefore the efficiency of a fallow period is limited. Crops grown will be dependent on the annual precipitation since little moisture will be stored from the year before. Fallowing on a high precipitation year with these soil conditions will rapidly recharge the water table in a saline seep, even if previously reclaimed. The other situation where annual cropping is adaptable is where the annual precipitation level is high, usually greater than 14 inches (35 cm) and the growing season precipitation is consistent and timely, 6 inches (15 cm) or better.

The flexible cropping system can be implemented almost universally. The decision to crop, made annually at seeding time is based on stored soil moisture plus expected growing season rainfall. Together, these must exceed 8 inches (20 cm) to achieve a satisfactory economic yield. The stored moisture is a measure of the depth of moist soil X the plant available water/ft. moist soil. The expected rainfall is the average amount received from May 1-July 31 at 70% probability or 7 out of 10 years. With this information, a yield prediction can be made on individual fields to assist the land operator in crop rotation decisions that are both economical and conservation minded. If the moisture conditions are insufficient, then a fallow year is inserted into the rotation. One guideline being never to fallow two years in a row to avoid any loss to deep percolation.

Intensive cropping systems require a high level of management by the operators. Careful attention must be paid to details like soil testing and fertilizer application, selecting the most suitable crop and cultivar, adequate weed control, marketing, etc. Alternate crops to cereal grains, specifically oilseeds, may be inserted into the rotation where the climate and market are suitable. The oilseeds will break up the monoculture to change weed and disease cycles inherent to small grains. Safflower and sunflower will be more moisture efficient than small grains (Table 2).

The economic feasibility of a flexible cropping system versus the traditional crop/fallow system can only be compared if accurate records are kept of variable costs and operations on a long term basis. It has been shown that with good management and marketing skills, the intensive cropping system can outproduce the crop/fallow system in both yield and net return (TCD unpublished data).

The discharge area treatment may not begin for several years after the recharge area cropping system has been intensified. As the ground water table is being lowered, natural precipitation will leach the precipitated soluble salts back down the profile. Soil tests of the top 3-6 inches (7-15 cm) are taken periodically to determine the electrical conductivity (EC). When the soil EC and water table have been reduced and an adequate seedbed can be prepared, salt-tolerant vegetation (primarily grasses) is established. The introduced vegetation will further lower the water table and provide weed and erosion control. It may be necessary to leave the seep area in perennial vegetation. But, in many cases, the seep can be reclaimed to original condition and again produce crops.

An example of a site specific saline seep reclamation plan involved a drainageway in Teton County leading into Priest Lake that had been salinized for a distance of 3 miles (4.8 km). In July, 1981, the drainageway flowed at 11.2 gpm (2.5 m^3/hr) with a TDS of 78,310 mg/l. The high sulfate content, 58,700 mg/l had caused severe deterioration of the concrete road culvert. The recharge area above one of the seeps was cropped in 1982, alfalfa was seeded in 1983 and a 2.5 Ton/ac (5.8 m.t./ha) hay crop was harvested in 1984. The water level in the seep dropped from 15 inches (38 cm) -'82 to 37 inches (94 cm) -'83 to 78 inches (198 cm) in 1984. Flexible cropping above other seeps has also dropped the water level, though not as dramatically. By July, 1984, the drainageway was no longer flowing. In contrast, another field was fallowed in '83, and the associated seep water level was raised 10 inches (25 cm) to surface level, indicating the delicate balance in recharge and discharge area water use.

Another reclamation success involved a total of 20 acres (8 ha) of salinized cropland in Toole County. Observation wells were installed in the recharge and discharge areas to monitor the shallow ground water tables. Approximately 90 acres (36 ha) of Ladak 65 alfalfa were seeded in a designated recharge area. Additional surrounding cropland was flexible cropped to increase overall water efficiency. In 1984, four years after plan implementation, the water table level in the recharge area had dropped from an original level of 63 inches (160 cm) to 178 inches (452 cm). Likewise, the water table in the discharge area dropped from its 1980 level of 34 inches (86 cm) to a 1984 level of 94 inches (239 cm). Figure 4 graphically displays the water table level decreases. Consequently, the annual precipitation has leached the salts lower into the soil profile allowing alfalfa to spread into formerly salinized areas.

Summary

Favorable hydrogeologic and climatic conditions for saline seep development exist throughout northern Montana. Inefficient moisture management in our current farming practices has aggravated the situation resulting in an extensive loss of arable land. The ramifications of this resource problem extend beyond the scope of salinized cropland. The deterioration in shallow ground water quality has, in several instances, necessitated the development of rural pipeline systems.

To prevent further ground water quality degradation requires preventing deep percolation of soil moisture and subsequent migration of soluble salts. The key to resolving this resource problem is an increased level of management by the landowner and/or operator. A hurdle present in any reclamation plan is the economic reality of everyday life. Conservation and reclamation have to pay if there is to be a widespread adoption of a plan or project. The TCD has been well received by local land managers due to our practice of tailoring each plan to suit the individual site's characteristics. Through the use of deep-roted perennials, i.e. alfalfa, and intensifying cropping systems, the TCD is observing significant decrease in both size of seeps and water table levels in as little as five years. As results begin to accumulate, interest has been stimulated in areas outside the 10 county Triangle Conservation District.

The knowledge gained in vegetation management for saline seep control is now being adapted for other water quality problems. The TCD is participating in a research project, in conjunction with the Montana Bureau of Mines and Geology, to determine the viability of intensive management as a key to cleaning up acid mine drainage. In the Stockett-Sand Coulee area of central Montana, inefficient moisture management on cropped benches has allowed soil moisture to percolate past small grain roots and into lower-level, abandoned coal mines. Although the problem differs in type of pollutant, the process leading to formation is the same.

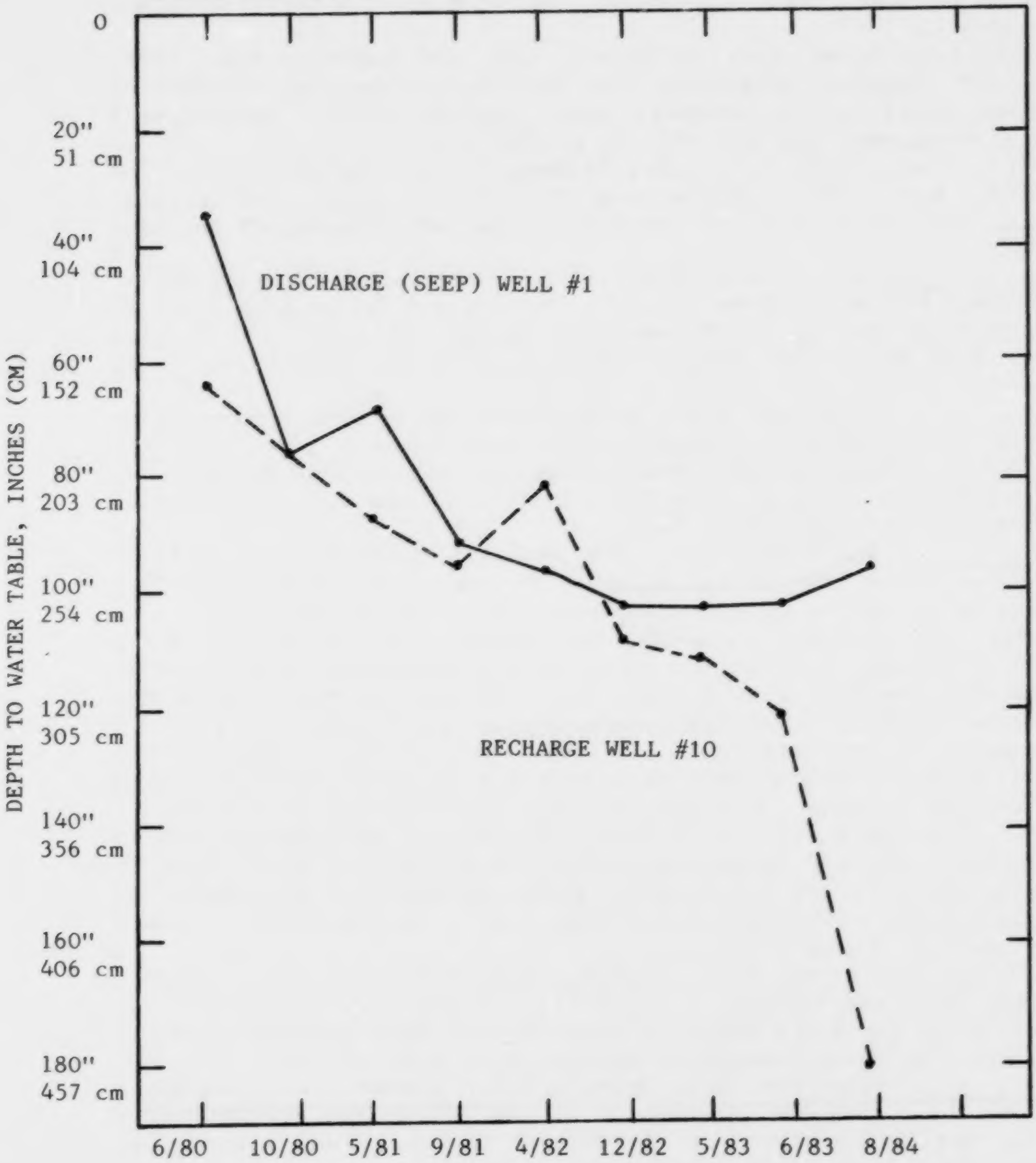

Figure 4. Effect of alfalfa in lowering ground water tables in the recharge and discharge areas on the Tomshx, Inc. farm near Shelby, MT. (NOTE: The spring meltwater (4/82), indicated by the peak in the recharge area water table, was fully utilized by the alfalfa before it could contribute to the discharge area).

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Session V
Dealing with Abandoned Wells

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ABANDONED WELLS - HOW TO FIND THEM

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Abstract

Improperly plugged or unplugged abandoned wells which penetrate an injection formation may provide a conduit for migration of injected fluids into fresh water formations. With the adoption of the Federal Underground Injection Control Regulations (UIC), all abandoned wells within an "area of review" around a proposed injection well must be located. This will help to ensure that if an abandoned well is present, the potential for contamination of the fresh water through the abandoned well is minimized.

A search for abandoned wells may have three different objectives: 1) to provide an overview of the presence or absence of abandoned wells within an area, 2) to determine the status of a particular well and establish the potential impact of that well, and 3) to actually field locate the abandoned well. The scope of a search may encompass all or any combination of these objectives before the search is completed.

To date, few methods have been successfully used to search for abandoned wells. Most searches have employed a combination of record searching, talking with residents, looking for the well and walking over the area with a metal detector or magnetometer. While few methods have actually been used, a variety of other technologies, although not specifically developed for this purpose may be useful in searching for abandoned wells. Geophysical methods such as electrical resistivity, electromagnetic conductivity and ground penetrating radar all may have various applications in searching for abandoned wells. Remote sensing techniques, including black and white aerial photographs, color photographs, color infrared and thermal infrared may be combined with other methods to provide a different dimension to the search. Other more indirect methods such as water-level measurements or actual injection may also be applicable in certain situations. The best combination of methods depends on the objectives of the search, the condition and surface expression of the abandoned well and the resources available to conduct the search.

Introduction

In the early days of oil and gas exploration, wells were often drilled **without** recording the location, depth or completion of the well. If the well was unproductive, the casing may never have been set, cut off below the ground or pulled. As time progressed, better records were kept of well location and production, but the locations were often nebulous or inaccurate and frequently could not be used to find the well accurately. In recent years, better records have been kept and detailed site locations are available for wells.

The magnitude of the potential problems from abandoned wells can best be assessed by looking at the number and the condition of abandoned wells. It is estimated that since 1859 when the first wells were drilled in Titusville, Pennsylvania until 1981, over 2,750,000 wells were drilled in the United States. However, in 1981, only 740,000 wells were producing oil and gas (Anonymous, 1982 a,b). This means that almost two million wells have been abandoned.

The way in which these wells have been abandoned has also varied through time. In early days, wells were frequently left open or filled with any nearby garbage or tool scraps. Tree trunks were also used as plugs to prevent livestock from breaking a leg when stepping in an open hole. As regulations developed and evolved, so too have the requirements for plugging so that what was acceptable and the latest technology in the 1940's may no longer be adequate today.

With the advent of underground waste disposal through injection wells and the use of wells for secondary recovery of oil by waterflood, abandoned wells were "rediscovered." Perhaps the most spectacular example of a problem directly traced to an injection well is cited by Hopkins (1963). Upon initiation of injection operations in a nearby oil and gas field, a gas furnace in the living room of a nearby farmhouse spouted brine. When injection operations were stopped, the flow of brine also ceased.

Although many problems associated with injection wells have not been as clear-cut or so easy to trace, the U.S. Environmental Protection Agency (U.S. EPA), in the promulgation of the Underground Injection Control Program (UIC), recognized the potential problem that could be associated with abandoned wells. The main thrust of the UIC is to assist in the establishment of effective state programs to protect underground sources of drinking water from the subsurface emplacement of fluids through well injection. Included in the minimum regulations of the UIC Program is the requirement that all abandoned wells within an "area of review" around the well be located and information regarding their completion and plugging be made available to the permitting agency. The permitting agency is then charged with determining if the potential for pollution exists from any of the abandoned wells.

The term "location" of abandoned wells may be interpreted either broadly or narrowly depending on the potential pollution threat. First, "location" can be providing an overview of an area to simply delineate the presence or absence of abandoned wells. Second, a more detailed "location" and well status could be established. Third, it may be necessary to "locate" the well in the field to actually reenter and plug the well. The type of search which is conducted is based on the degree of "location" which is necessary.

Abandoned well searches may be conducted using a variety of methods. These methods may be employed singularly or in combination to reach the level of "location" which is necessary. The paper provides a brief summary of methods which have been used in the past and methods which may have applicability for searches in the future.

Record Searches

Record searches are the logical place to begin any type of search for abandoned wells. Records are available from a variety of sources including: state regulatory agencies, private companies, county tax maps, leasing agreements, deeds and service companies. The information may be available in a variety of forms and may be accessed using a variety of systems. For example, some states place well locations on maps for easy reference (Figure 1). These maps contain production information and have a number which can be cross referenced with drilling and completion reports. These types of maps may not yield information on the status of the well, but certainly would provide overview information for an area.

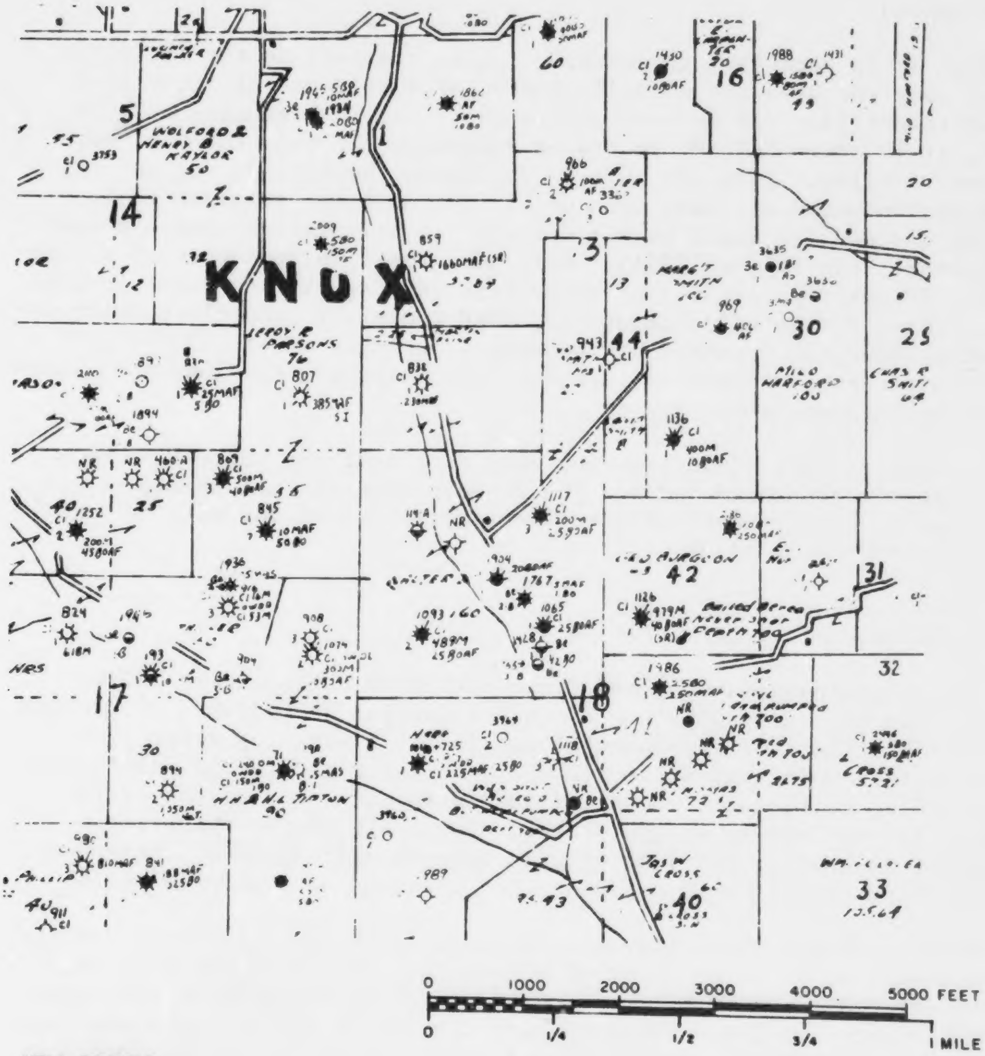
In addition, specific plat maps may assist when field location of a well is necessary. Plat maps vary in legibility and accuracy depending on the requirements of the regulations and the ability of the draftsman. Engineered, scaled plat maps are frequently available for new wells, while less professionally drafted and harder-to-read maps may be available for older wells.

In general, a search of records can be extremely valuable because it can provide a starting place to search for both cased and uncased wells and may be all that is necessary in the search. Conversely, well records may be incomplete or unavailable and well locations may be inaccurate.

Conversations with Local Residents

Local residents may be able to provide a great amount of information about well drilling activities in an area. Older residents and former oil field workers may have first-hand knowledge of the actual location of wells and often may be able to pinpoint the old well. Younger residents may also know of drilling activities through stories that were recounted to them by older relatives who may now be deceased.

This frequently overlooked source of information may be used to locate either cased or uncased wells, may reduce search time, may pinpoint the well location and may provide information which may not be available from other sources. However, the method may be time consuming and the residents may not know the desired information.


KNOX
0
1000
2000
3000
4000
5000 FEET
0
1/4
1/2
3/4
1 MILE

WELL SYMBOLS

- ○ location
- ⊘ abandoned location
- ☼ gas well
- ☼ gas well; show of oil
- ☼ abandoned gas well
- ● oil well
- ✸ oil and gas well
- ✸ oil well; show of gas
- ● abandoned oil well
- ⌖ dry hole
- ☼ dry hole; show of gas
- ⌖ dry hole; show of oil
- ☼ dry hole; show of gas and oil
- ❂ production from two or more horizons

Figure 1. Part of a township map illustrating typical platted information for more recent wells.

Visual/Logical

When it is necessary to actually locate the well in the field, walking over the area and visually looking for evidence of activities related to drilling may be very productive. Before beginning a field search, it is often helpful to try to determine the year the well may have been drilled. This may help to determine the type of rig and rig configuration which may have been used. For example, a wooden cable tool rig and derrick would have a different configuration than a steam driven rotary rig of the 1930's. Soil discoloration, evidence of filled-in pits, depressions created by heavy rigs or derricks, concrete foundations, roads, scraps of cable or discarded tools all lend clues which can be used to find the location of the well. Johnston et al. (1973) have described in detail things to look for when searching for abandoned wells in the Appalachian area.

In general, all field searches rely on visual and logical methods to help locate abandoned wells. The disadvantage of this method is that there must be some recognizable surface expression of the well to actually pinpoint the location.

Aerial Photographic Interpretation

Aerial photography has been flown over many areas of the United States since the early 1930's. Historical aerial photographs are available from a variety of governmental sources at a very low cost. The United States Geological Survey (USGS) National Cartographic Information Center (NCIC) maintains a computerized listing of the available aerial coverage for the entire United States. The scale of the photographs varies but in general any imagery with the scale 1:40,000 or larger may be able to be used to help delineate the location of abandoned wells.

Historical aerial photographs may actually be used to "see" drilling features such as derricks, central powerhouses or rod lines or to see pits or other surface features associated with drilling which may have been obliterated on the surface (Figure 2). Stout and Sitton (1984) have described a methodology used to identify well locations by developing "signatures" which serve as indicators of well locations. These signatures when developed through time can help pinpoint the location of the well.

In general, aerial photographic interpretation can be used to locate either cased or uncased wells. The photographs may actually show surface disturbances which have since been obliterated and may show features which are no longer evident at ground level. However, photographs may not be available for all years in the area and the location of wells on the photographs must be field checked in order to verify that they truly exist.

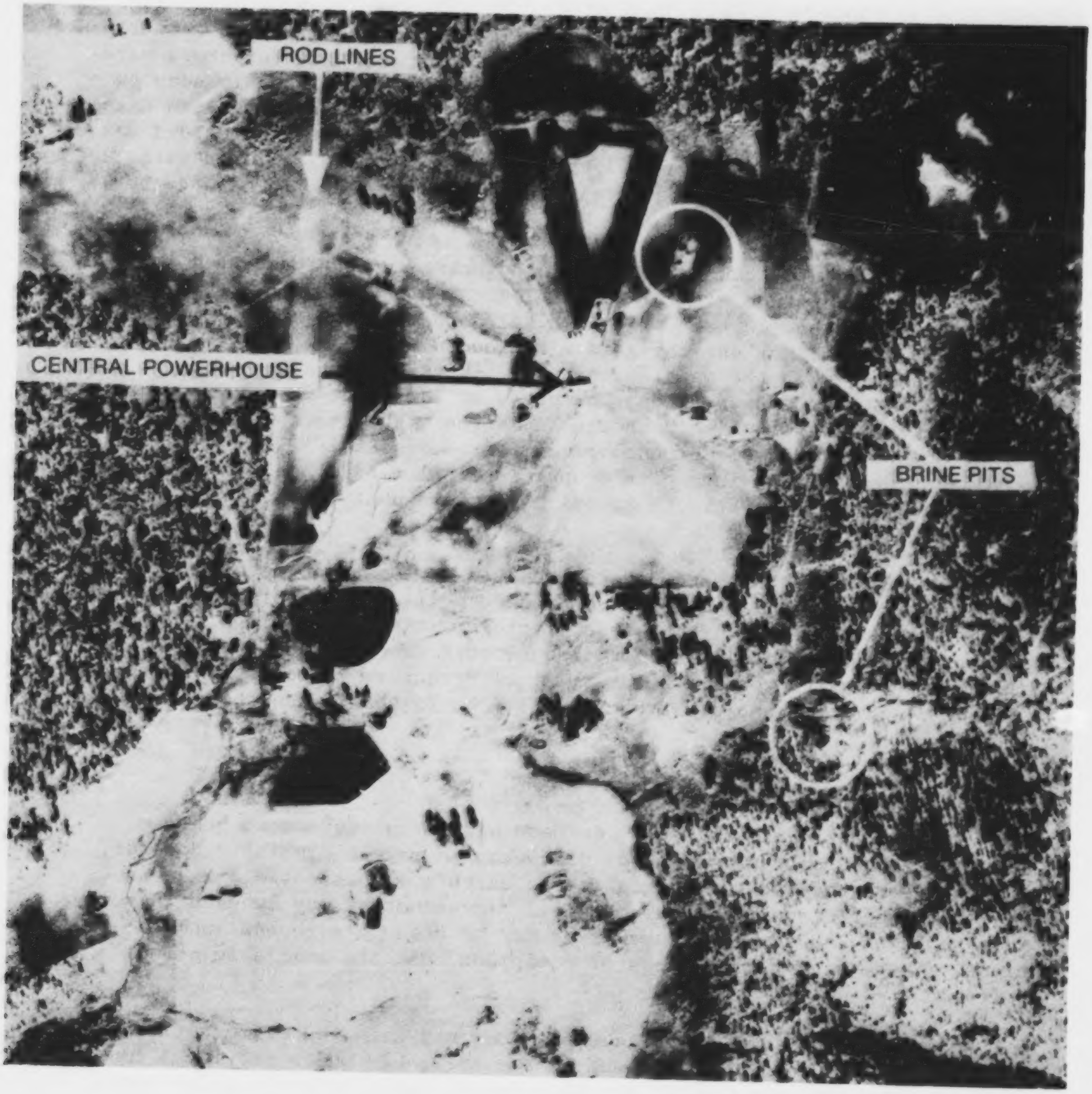

ROD LINES
CENTRAL POWERHOUSE
BRINE PITS

Figure 2. Aerial photograph showing central power house, rod lines to the power house and brine pits, Osage County, Oklahoma, 1937.

Metal Detectors

Once the preliminary methods such as record searching or examining aerial photographs has been accomplished, it may be necessary to actually field locate the wells. In this case, tools such as metal detectors may prove very useful. Metal detectors are designed to continuously scan an area for metallic objects. Traditionally, metal detectors have been used by treasure hunters to find buried metallic objects. Metal detectors may also be used to find buried metallic casing or objects associated with drilling. The objects must be metal, but do not necessarily have to be ferrous metal.

A metal detector can be used to help locate an abandoned well by using it in combination with visual or logical clues to help indicate where the search should take place. Metal detectors may also be used by setting up a grid, marking the grid with wooden stakes so that there is no interference with the metal detector, walking over the ground in a systematic fashion and operating the piece of equipment in a sweeping motion. Locations of objects should be marked with wooden stakes and either dug up immediately or uncovered later. Because drilling objects were frequently thrown into pits or put into piles and then later covered over, it is not always safe to assume that the density of objects gets greater closer to the well.

In general, metal detectors can be used to locate objects associated with drilling and to locate cased abandoned wells. The equipment is fairly inexpensive and portable and little training is required to be able to operate the equipment. Metal detectors can be used in all terrain and most vegetative cover. However, metal detectors may only assist in finding cased wells and are typically limited to shallow depths.

Magnetometers

Magnetometers are commonly confused with metal detectors because the equipment appearance and mode of operation may be similar. However, magnetometers respond to changes in the earth's magnetic field and can be used to detect only ferrous metals. Magnetometers may be used in surface searches, subsurface searches or may be used in airborne applications. There are various types of equipment that are available at all prices.

Magnetometers used in surface searches may have two different types of data output. First, magnetometers are available which provide a continuous output in the form of a tone. Second, magnetometers which provide discrete readings of the earth's magnetic field at specific points may be used. Searches can be conducted by using equipment with either type of output by either setting up a grid and taking readings along the line or walking along with a continuously recording piece of equipment until objects are found. Ideally, the response of equipment which provides discrete readings should look very similar to Figure 3 when the readings are plotted. Well casings plot as a symmetrical peak when taken in a north/south direction.

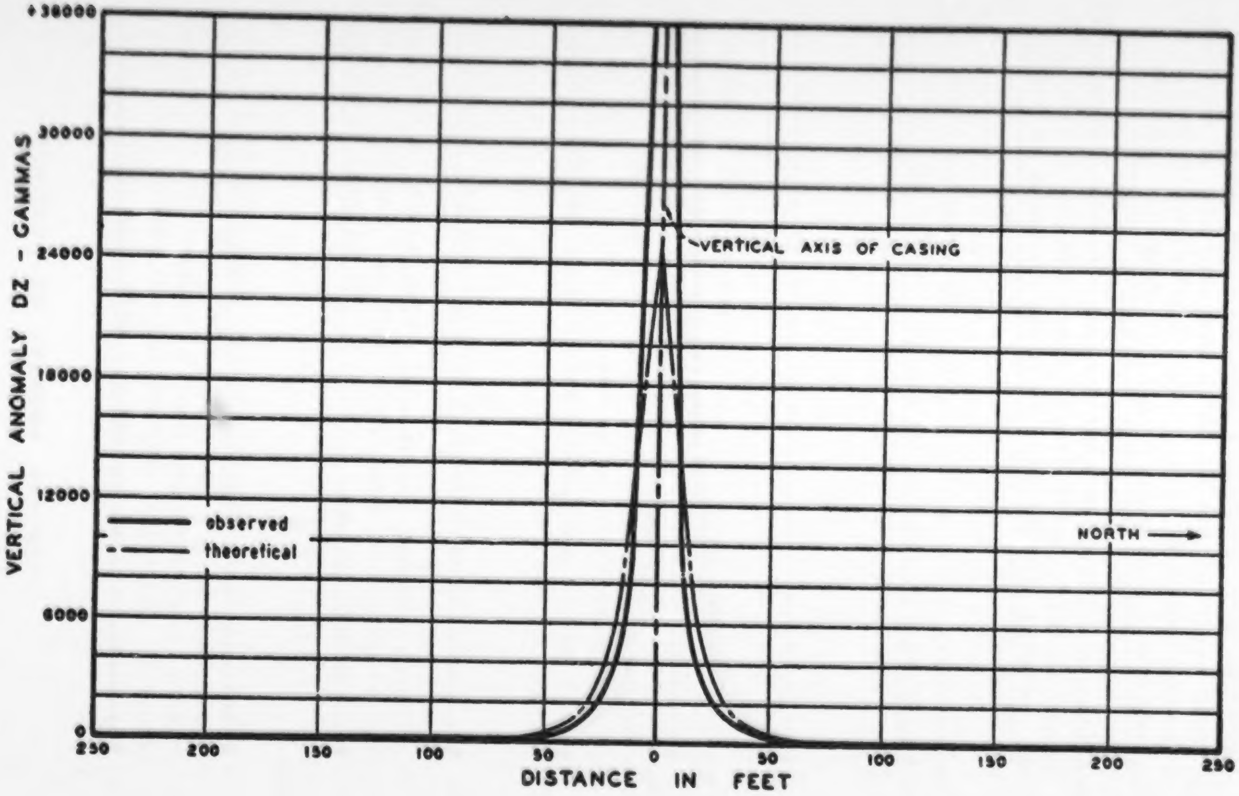


Figure 3. Comparison of observed and theoretical anomaly produced by a 4,609 foot vertical string of casing (Barret 1931).

In general, however, surface searching magnetometers offer many advantages when searching for abandoned wells. The equipment is portable, can be used in all terrain and vegetative cover, little or no training is necessary to operate some of the equipment and usually this equipment requires little or no interpretation.

Magnetometers may also be used aerially by mounting equipment on an airplane and using field verification to check the location of the abandoned wells. Frischknecht and Raab (1984) have preliminarily tested airborne equipment in searches for abandoned wells in Oklahoma and have suggested that this method may be promising for use in other areas. Aerial searches are limited to less populated areas, however, because the searches require fairly closely spaced flight lines and low flight altitudes. In addition, readings are affected by other cultural features which must be accounted for.

In general, magnetometers may be very useful in locating cased wells. However, they are traditionally limited to shallow depths and some equipment may require interpretation, especially when used in airborne application.

Combustible Gas Indicators

Combustible gas indicators have limited application for locating abandoned wells. The equipment is available from a variety of sources and is designed to detect and measure combustible gases in the air such as methane. This method can be used where there is a minimal surface expression of a well but where there is an open conduit from the formation to the surface. If gas is associated with the old production formation, the gas may migrate to the surface in great enough concentrations to be detected.

A search for an abandoned well using a combustible gas indicator can be conducted by setting up a grid of the area and taking into account the wind direction. The equipment is typically operated by walking over the ground and slowly sweeping the instrument back and forth. If gas is present, an audible alarm will sound. Generally the highest concentrations of gas should be closest to the well when the wind direction is taken into account.

In general, combustible gas indicators have been used to locate cased and uncased abandoned wells. The equipment is portable, inexpensive and easy to operate. However, gas must be present and detectable and must have a conduit to reach the surface for this method to be used. In addition, wind may disperse the gas and may make this method not viable on windy days.

Excavation

Excavation is the final procedure used to verify the presence or absence of evidence of a casing, wellbore or objects associated with drilling and production activities when there is no or only a limited surface expression. Excavation may be accomplished using something as simple as a shovel or by using a bulldozer to move large volumes of earth. Excavation may finally prove or disprove the location of an abandoned well. However, large areas may be excavated without actually finding the well location.

Electrical Resistivity

Electrical resistivity is a geophysical method which is designed to measure the apparent resistivity of earth layers. Traditionally, it has been used in mineral exploration. It was hoped that electrical resistivity might be applicable to finding metal casing present in abandoned wells. A electrical resistivity survey is conducted by injecting electrical current into the ground through two current electrodes and measuring the current drop along two potential electrodes. All four probes must physically be inserted into the ground and a reading must be recorded. The probes are then removed from the ground, placed in another spot and another reading is taken. Once the data is obtained, it must be interpreted by someone who is familiar with the technology.

Although it was hoped that electrical resistivity might have application to finding buried metallic casing, because the electrical anomaly produced by a casing is so small, it may not be able to be detected. Electrical resistivity may still be able to be used, however because recently electrical resistivity has been used to track contamination plumes with high specific conductance. The best application of electrical resistivity may be actually tracing a source of contamination back to a well which may be abandoned.

In general, electrical resistivity may be used to help find and delineate a saline contamination plume and trace it back to a buried abandoned well either with or without casing. However, operator expertise is needed when conducting electrical resistivity surveys and data interpretation is necessary once all the information is collected. The electrodes must physically be placed into the ground and this makes this geophysical method fairly slow. In addition, there are limitations in in steep terrain and vegetative cover because the readings must be taken in a relatively straight line and must be strung together with cable to the electrical source. Additionally, electrical resistivity is expensive when compared to other geophysical methods for this particular application.

Electromagnetic Conductivity

Electromagnetic conductivity is another geophysical method which measures variations in the conductivity of the earth. Using a transmitter, a magnetic field is generated which induces a second magnetic field in the earth materials. The result is detected at the receiver as a ratio of the primary and secondary fields and displayed as a reading. The equipment is portable and is operated by walking over the surface of the ground, typically in a grid fashion. Originally it was hoped that electrical conductivity could be used to find cased abandoned wells which had been cut-off below grade. However, as with electrical resistivity, this method is probably best applied to finding and tracing saline ground water contamination plumes back to their source.

In general, electromagnetic conductivity may be used to trace saline contamination plumes back to abandoned wells which may be causing a problem. Electrical conductivity may also be used to find soil disturbances larger than a borehole such as would be associated with brine pits. The equipment is portable, not limited to shallow depths and can be used in all terrain and vegetative cover. However, special expertise is needed to operate the equipment and the equipment is expensive. Once the abandoned well location has been inferred, then other methods are still needed to verify the actual abandoned well location.

Ground Penetrating Radar

Ground penetrating radar is another geophysical method which uses high frequency radio waves which are transmitted into the ground and reflected back to be recorded at the surface. Ground penetrating radar output is typically in the form of a "picture" which is displayed on a graphic recorder at the surface as the equipment is either hand towed or truck towed over the surface of the ground (Figure 4). Ground penetrating radar has been most extensively used in hazardous waste disposal sites to locate objects such as buried drums. However, ground penetrating radar is also applicable for locating leachate plumes and small soil disturbances. It is hoped that ground penetrating radar may be used to find soil disturbances as small as those which would be associated with an abandoned well. In actuality, ground penetrating radar has been successfully used to find an abandoned well when conducting a survey for other purposes (McKown and Sandness, 1981).

In general, ground penetrating radar may have broad application because it can be used to find either cased or uncased abandoned wells. Ground penetrating radar may be used to find soil disturbances associated with drilling activities or to actually find an uncased wellbore. The equipment provides continuous readings in the form of an on-site graphic recorder which helps to provide on-site interpretation. Depth penetration is typically 10-25 feet. The disadvantages of using ground penetrating radar is that it is relatively expensive, and since the equipment must be towed over the ground, there must be access for towing. This limits ground penetrating radar to certain types of vegetative cover and terrain. A professional company is needed to perform the survey and the equipment must actually pass over the well to find it. If the equipment does not pass directly over the well, the disturbance due to the abandoned well will not be detected.

Remote Sensing

Remote sensing is used to gather data about the surface of the earth using aircraft or satellite-mounted sensors. Remote sensing includes black and white aerial photographs, color infrared photographs and thermal imagery. Color infrared imagery, typically referred to as false color imagery, responds to the differences in the amount of radiation reflected

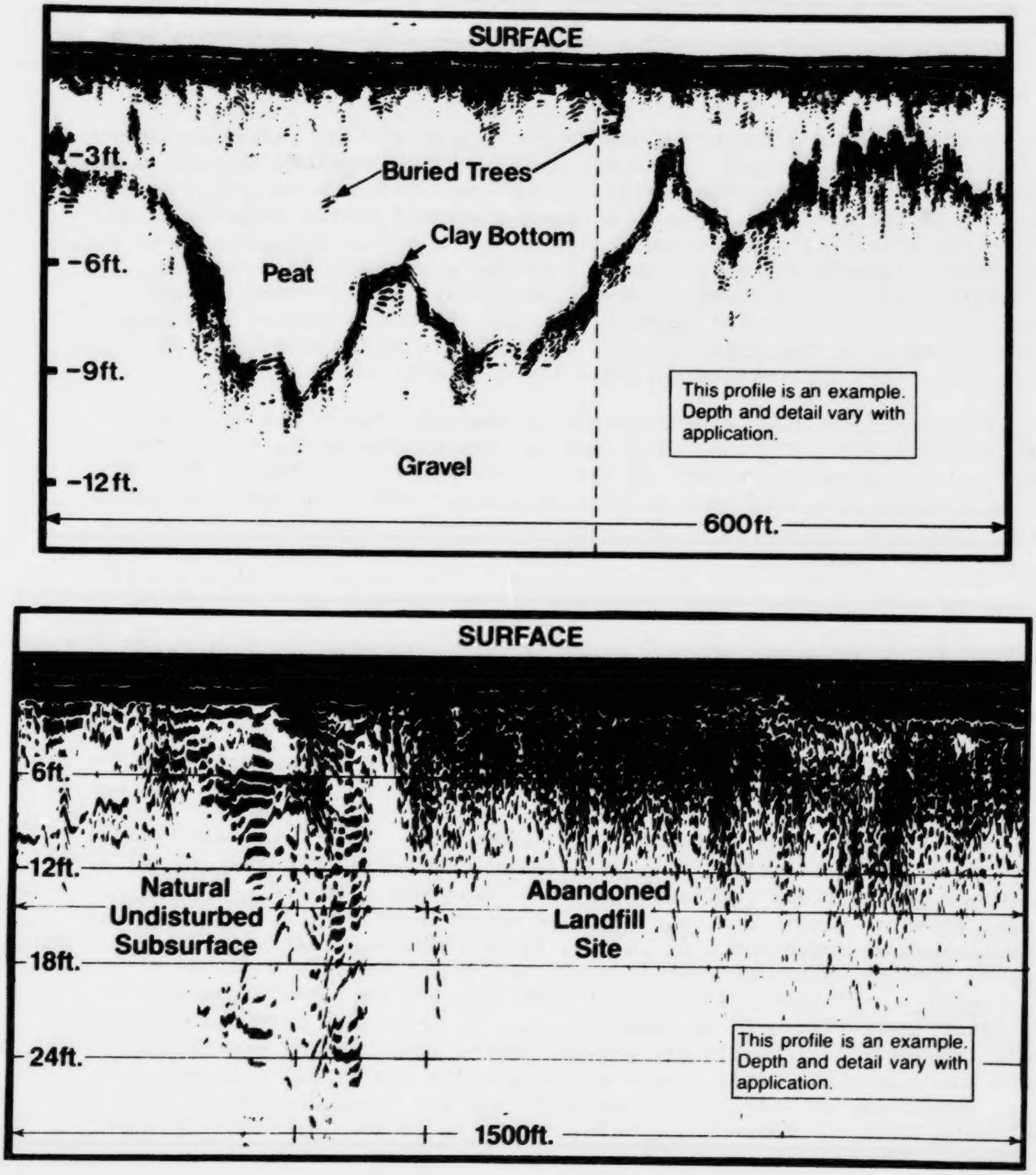


Figure 4. Example profiles obtained from a ground penetrating radar survey (Geophysical Survey Systems Inc.).

by objects being photographed. Color infrared film has typically been used to help determine vegetative stress in healthy deciduous foliage. If the vegetation is distressed or diseased, the reflectivity of the leaves decreases. This small color difference is not visible to the naked eye, but is visible through color infrared photography. Although this particular method cannot be used to find an abandoned wellbore, it could be used to find stressed vegetation that has been affected by brine contamination. This may help to imply the location of an abandoned well.

Thermal infrared imagery measures the amount of heat radiation that is emitted from the ground. Although this particular method has not been applied to locating abandoned wells, it is hoped that the natural thermal gradient of the earth may produce a thermal anomaly at the location of a well. The resulting picture would show a "hot spot" or bright point on the thermal imagery where the location of the well would be inferred. This partiuclar method shows promise, but nobody has performed thermal imagery for this particular purpose. Because of the necessity of flying thermal imagery at the right time of day and only at certain times of year, this particular method may have limited application.

In general, either color infrared or thermal imagery may be used to help locate abandoned wells. Both may be used to show surface features which are not normally evident at the surface and color infrared imagery may be used to help find vegetative stress associated with abandoned wells long before the contamination is found in any well. The disadvantages of using remote sensing may be that the imagery may not be as readily available as black and white aerial photography. This requires that a special survey be flown over the area of interest. The survey may be expensive and interpretation of the photography will be necessary once the survey is completed. In addition, the locations of any wells that are indicated on the photography must be field checked in order to determine whether a well truly exists at that site.

Water Level Measurement in Surrounding Wells

In areas where the hydrogeology is known, abandoned wells may be able to be found by using water level measurements in surrounding wells. Since the water level in any confined aquifer is an expression of the hydrostatic pressure within that aquifer, the water level typically responds to stimuli such as injection. In theory, if an abandoned well provides a conduit from an injection zone into an aquifer, a pressure build-up will be expressed as a water level rise (Figure 5). In an ideal situation, the water level rise will be greatest near the abandoned well and decrease radially outward. If wells are present in the area and the completion, depth and historical water levels of those wells are known, this particular method may be able to be used to locate the abandoned well. This particular method requires no special expertise to take water level measurement. However, the method may be time consuming.

In general, water level measurement in surrounding wells may provide a method of locating either cased or uncased abandoned wells with the use of no specialized equipment. However, local hydrogeologic information and existing wells must both be available for this particular method to be

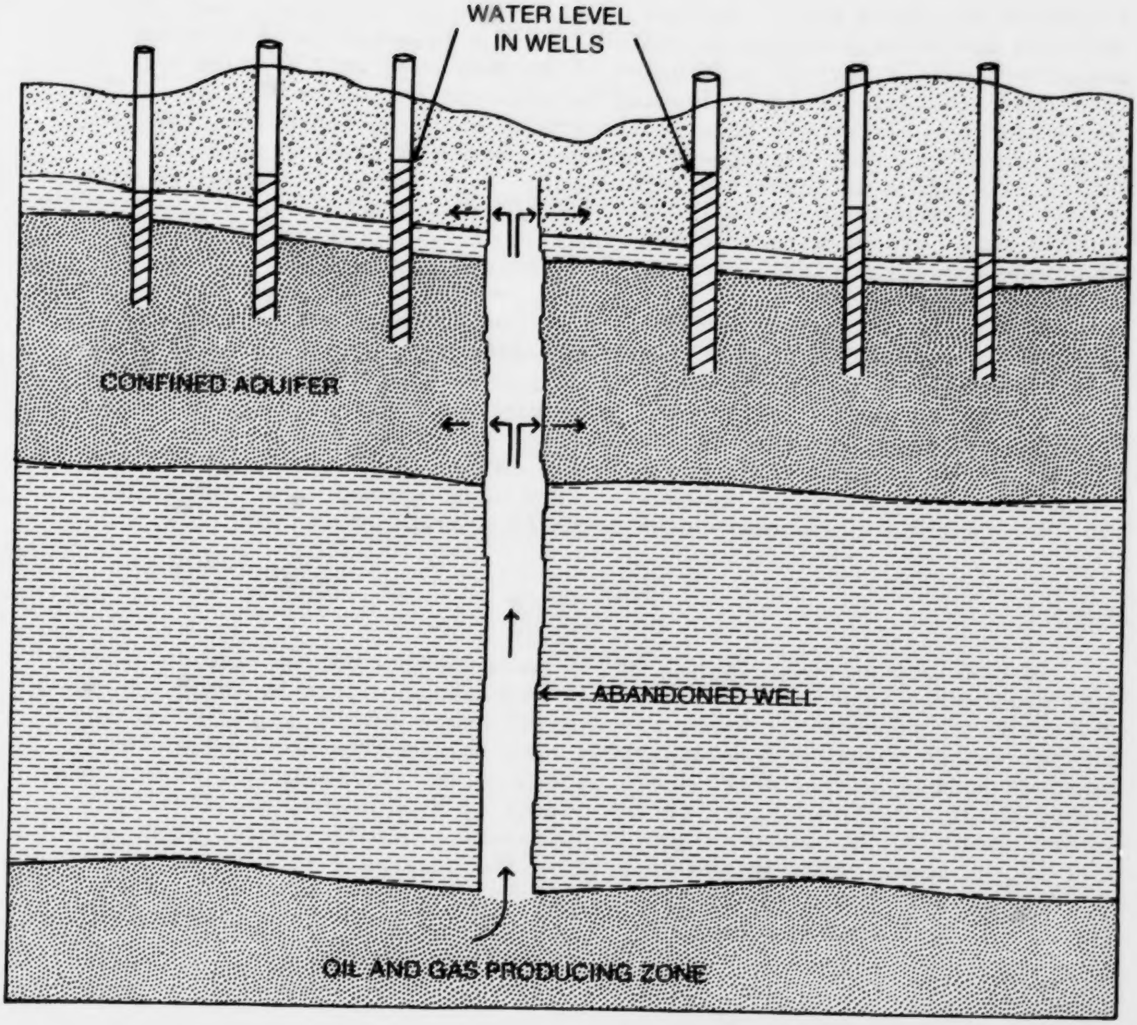

WATER LEVEL
IN WELLS
CONFINED AQUIFER
ABANDONED WELL
OIL AND GAS PRODUCING ZONE

Figure 5. Diagram illustrating water level increases in wells surrounding an abandoned well.

viable. Field location of the abandoned well will still be necessary because this method will only provide a general idea where an abandoned well might be located.

Injection

Injection is not a method which is recommended to actually field locate an abandoned well. However, where injection operations are present and where the pressure is sufficient, the injection of fluids may actually create a surface expression of an abandoned well (Figure 6). In this case, the well could be located by noticing the water flowing at the surface or by observing a hole in the ground when the injection operation is stopped. In other situations, however, actual migration might be taking place from an injection zone into an aquifer but the pressure may not be large enough to cause the fluid to flow to the surface (Figure 7). In this case, ground water contamination could be occurring but would go unnoticed until the plume intersected a water supply well.

In general, injection can be used to locate cased and uncased abandoned wells especially when surface expression of the well is created when injection operations are started. The disadvantages in using injection to locate abandoned wells is that the pressure must be great enough to create fluid migration to the surface, the channel to the surface must be well defined enough so that fluid can make its way to the surface and the contamination may not show up immediately and cause ground water contamination before the surface expression actually occurs, if it ever does.

Summary

In conclusion, there is no one set method which can be used to locate abandoned wells in all situations. A search should normally begin by conducting a record search. Next, the project scope should be fully defined and the search objectives established. The search area should then be narrowed using whatever methods are necessary and the most cost effective method should be chosen. Before the search is even begun, the level of effort should be evaluated so that unnecessary expenditures of resources will not be made. And, in the end, it may be necessary to give up without finding the abandoned well because it is simply not cost effective.

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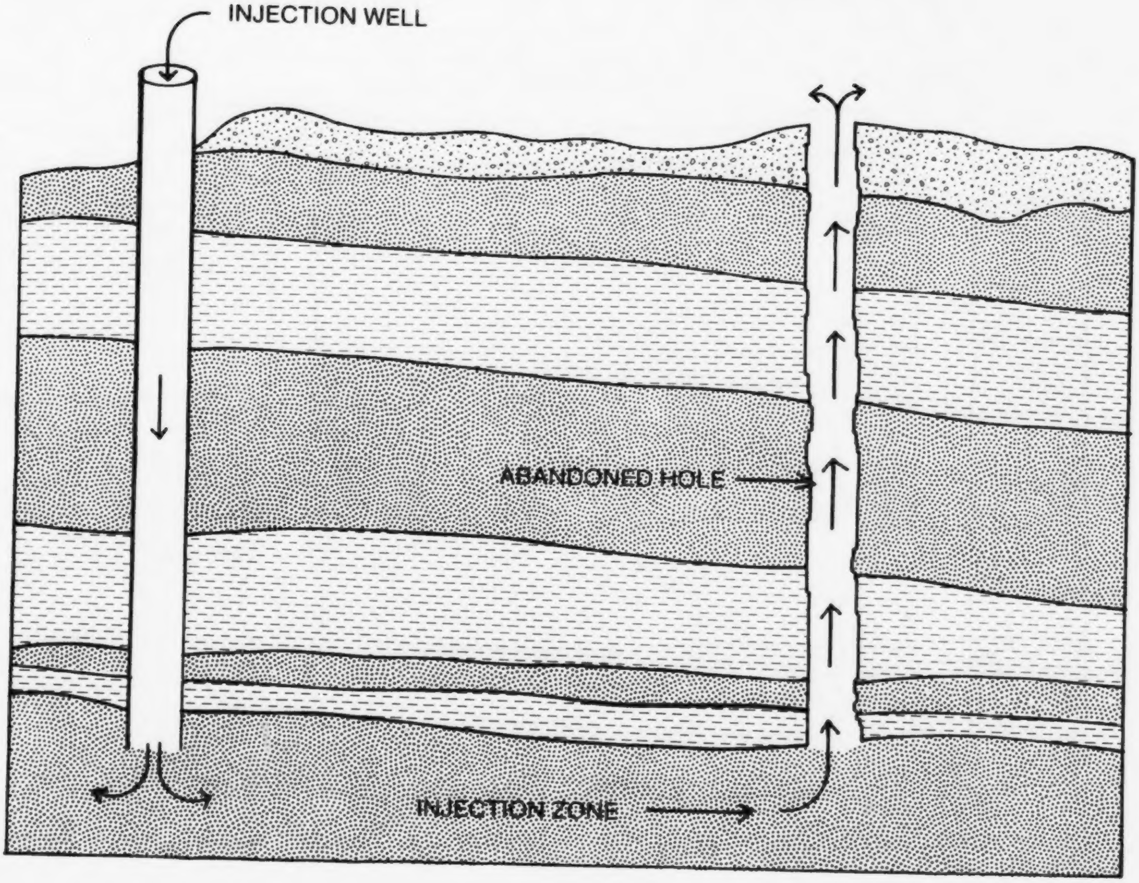
INJECTION WELL
ABANDONED HOLE
INJECTION ZONE

Figure 6. Diagram of the relationship between an injection well and a flowing abandoned well.

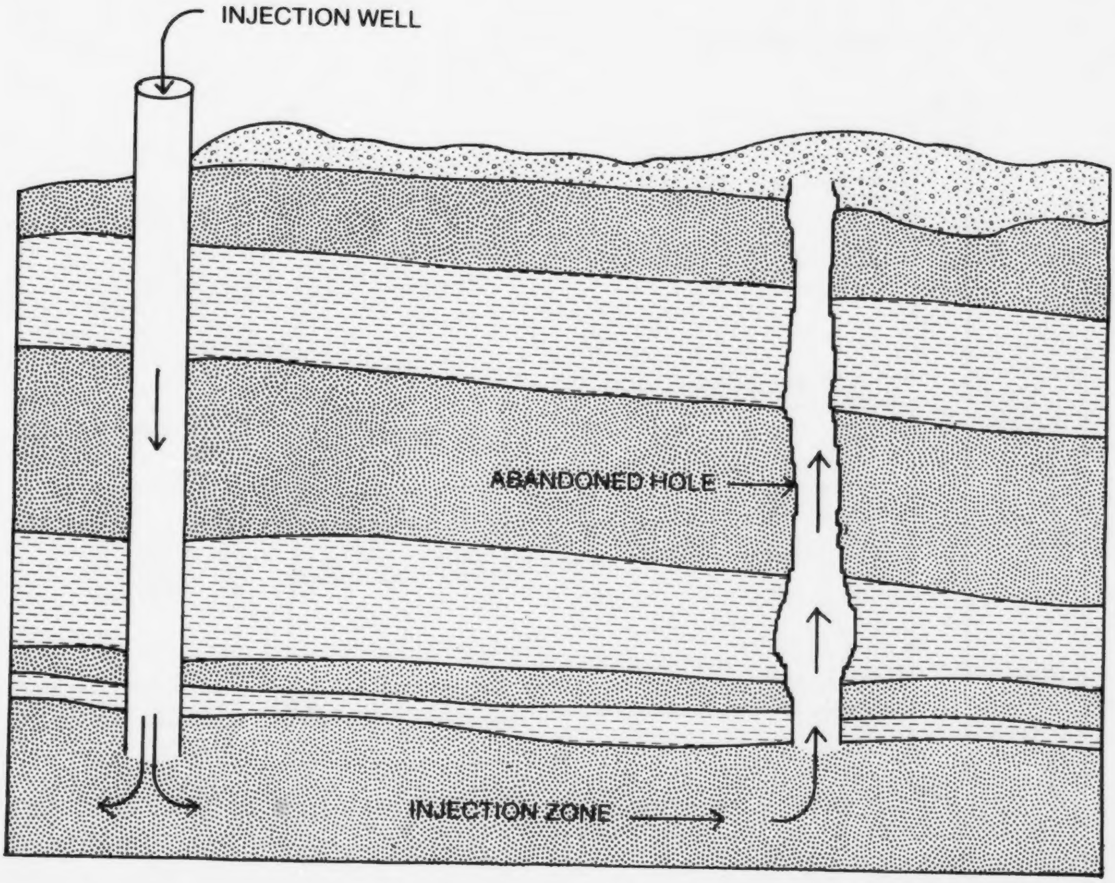
INJECTION WELL
ABANDONED HOLE
INJECTION ZONE

Figure 7. Diagram of the relationship between an injection well and an abandoned well which does not flow at the surface.

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Biographical Sketch

Linda Aller, a research hydrogeologist for the National Water Well Association (NWWA) holds a B.A. in geology from the Ohio State University and has done postgraduate work in hydrology at both Ohio University and the Ohio State University. Prior to joining NWWA's research staff, Mrs. Aller was the principal geologist responsible for the development of Ohio's private water system rules including those rules governing well construction. Since joining NWWA, she has been involved with many research projects including deep well injection practices in the petroleum industry and aquifer restoration. Mrs. Aller is a registered sanitarian in the state of Ohio.

SUMMARY AND COMPARISONS OF THREE TECHNOLOGIES FOR LOCATING ABANDONED WELLS IN CENTRAL OKLAHOMA

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ABSTRACT

The U.S. Environmental Protection Agency evaluated three techniques for locating abandoned oil and gas wells in twenty-four sections of four townships near Oklahoma City, Oklahoma. The techniques were a traditional search of the records, an historical aerial photography search and an aerial magnetometer survey. The University of Oklahoma's Environmental and Ground Water Institute performed the records search and selected the twenty-four sections for study. The Agency's Environmental Photographic Interpretation Center used historical aerial photographs from the 1930's through 1981 to locate wells in the study areas. The United States Geological Survey determined from mathematical modelling that high resolution aerial magnetometry surveys were capable of locating abandoned wells, and conducted flights at 61-meters altitude and 100-meter spacings in Oklahoma. Approximately 95-98 percent of the probable number of oil and gas wells were located by the magnetometer survey compared with 91 percent from the photographic search.

The advantages, disadvantages, and costs of the three methods are summarized and suggestions are offered on how the methods may be applied to different areas.

Introduction

The Environmental Protection Agency (EPA) funded an assessment study of three of the most promising techniques for locating abandoned oil and gas wells near Oklahoma City. Record searches, historical aerial photographs, and magnetometer surveys were evaluated for their cost and effectiveness in locating wells. Searches of the Oklahoma Corporation Commission's records were conducted by the University of Oklahoma's Environmental and Ground Water Institute to locate oil and gas wells in 24 sections where aeromagnetic and ground-based magnetometer surveys could be conducted. Historical aerial photographs were used by the EPA's Environmental Photographic Interpretation Center (EPIC) to evaluate the method and to substantiate the well locations from the records search. The aeromagnetic survey and ground-based magnetometer measurements were made by the United States Geological Survey (USGS) to determine if wells with steel casing could be easily and precisely located and to determine the percentage of wells that would be missed because there was no casing or only short lengths of casing left in the ground.

Four separate test areas encompassing 24 square miles were selected by the Environmental and Ground Water Institute for study (Figure 1). Data were assembled for 493 wells in an area where drilling had occurred since 1917. Historical aerial photographs dating back to 1937 were obtained through a search of federal film repositories. Tables 1 and 2 show the dates, scale, and sources of photographs used in the study. The aeromagnetic survey was conducted in 1984 after mathematical modelling by the USGS indicated the feasibility of locating steel-cased wells with a low-altitude, high-resolution aeromagnetic survey. Ground-based magnetometer measurements were made at selected locations where a magnetic anomaly could not be associated with a recorded well location. In those instances where a well was actually located with the ground-based magnetometer and no evidence of the well could be seen on the surface, it was estimated that the location of the well bore could be pinpointed with an accuracy of 3 to 6 feet. This is in contrast to the 25 to 50 foot resolution achievable from historical aerial photographs and the 1/64-section or worse resolution obtained from a search of the records.

The Records Search

The number and location of wells identified from the records search could not be used as the sole basis for a comparison of the methods. Incomplete records, wide variation in well location information, and lack of confidence in well status determinations were some of the problems.

Many of the injection wells could only be located as being somewhere within a section. Other wells could be located anywhere from within a section to 1/64 of a section (10 acres). In some instances the coordinates for a well were transposed, thus, a well recorded at NWSWSE proved to be at SESWNW. (These errors were discovered when a check was made against the information obtained from the historical aerial photographs.) While the location of wells from the records was a time-consuming process with varying levels of accuracy, determination of the status of the wells proved to be even more difficult. All completed petroleum-related wells are supposedly recorded on completion reports. However, in a few cases plugging records were found for wells that had no completion reports. The production, abandoned, and active status reports covered time periods ranging from only a few weeks to as long as one year. In addition, the dates for this information varied from 1981 to 1983. Thus, determining the exact number, placement, and status of wells at the time of the aeromagnetic survey proved to be a tedious task with some uncertainty in the accuracy of the data.

Historical aerial photographs and magnetometer surveys to locate abandoned wells represent new methods that regulatory agencies could require to increase the positional accuracy and number of located wells.

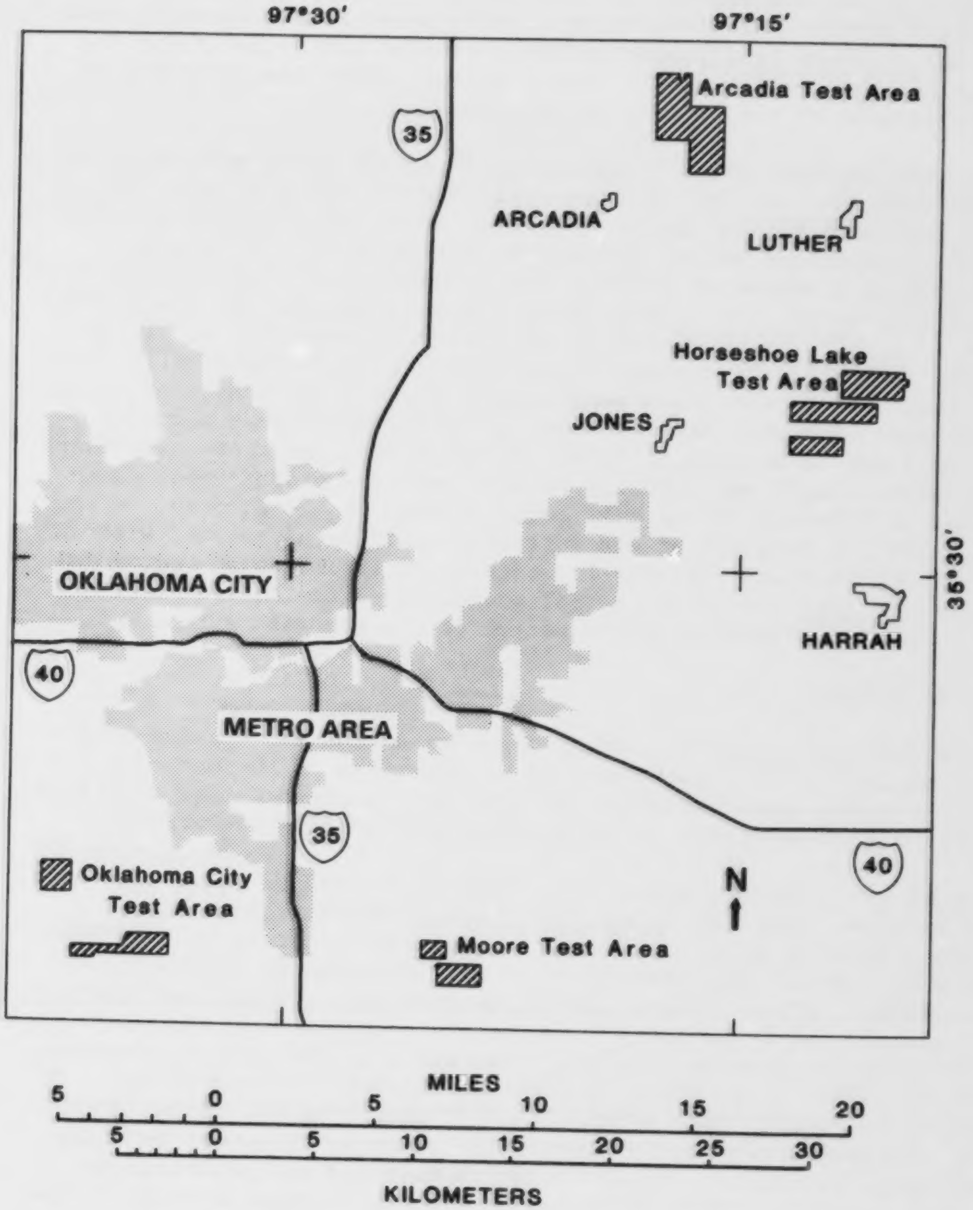

97°30'
97°15'
35°30'
Arcadia Test Area
ARCADIA
LUTHER
Horseshoe Lake Test Area
JONES
HARRAH
OKLAHOMA CITY
METRO AREA
35
40
Oklahoma City Test Area
Moore Test Area
N
MILES
5 0 5 10 15 20
5 0 5 10 15 20 25 30
KILOMETERS

Figure 1. Location of the Oklahoma test area.

Table 1. Aerial Photographs Used for Arcadia and Horseshoe Lake

Date	Scale	Source
August, 1937	1:80,000*	NARS[1]
October, 1941	1:80,000	NARS
March-April, 1951	1:20,000	ASCS[2]
July, 1957	1:20,000	ASCS
April, 1963	1:20,000	ASCS
December, 1969†	1:20,000	ASCS
January, 1970††	1:20,000	ASCS
January 1975	1:40,000	USGS[3]
April, 1979	1:40,000	ASCS

*Reduced from original scale of 1:20,000

†Horseshoe Lake only

††Arcadia only

[1] National Archives and Records Service, U.S. General Services Administration

[2] Agricultural Stabilization and Conservation Service, U.S. Department of Agriculture

[3] United States Geological Survey, U.S. Department of Interior

Table 2. Aerial Photographs Used for Moore and Oklahoma City

Date	Scale	Source
August, 1937	1:80,000*	NARS
March, 1951	1:20,000	ASCS
July, 1957†	1:20,000	ASCS
April-May, 1963	1:20,000	ASCS
September-December, 1969	1:20,000	ASCS
January, 1981	1:40,000	ASCS

*Reduced from original scale of 1:20,000

†Oklahoma City only

Neither of the two methods can completely replace the records search because only the records search can provide information on the plugging and mechanical integrity of wells to assess the potential for contamination. However, if problems similar to those encountered in Oklahoma are found in other states, the well locations obtained from a records search may be incomplete or inadequate.

Historical Aerial Photographs

The historical aerial photographic search used multiple years of historical aerial photographs to locate oil and gas wells when they were active and easier to identify and to trace their activity through time until they became inactive and abandoned. The four test areas: Arcadia, Horseshoe Lake, Moore, and Oklahoma City were photographed from 5 to 8 times between 1937 and 1981.

The first step in locating oil and gas wells on aerial photographs was to develop well site "signatures". A signature is a combination of characteristics or features by which an object or activity can be identified on a photograph. Details on the development of the signatures can be found in the literature or provided by the EPA's Environmental Photographic Interpretation Center (EPIC).

Analysts found that signatures varied from one area to another, depending on the time the oil and gas field was developed and the technology which was used. For instance, several wells were present in the Moore study area by 1937 (see Figure 2). The drilling technology employed in that area during that time period results in a very uniform well site configuration consisting of three brine pits, a derrick foundation, and a small power house at each well location. Figure 3 shows the same area as it appeared in 1981. All that remains of the old wells are a few revegetated impoundments and some of the derrick foundations; several well sites have been completely obliterated.

Well sites in Arcadia exhibited a slightly different signature. Figure 4 shows an area that was active in 1951. The well locations were identified by the presence of either a derrick or a type of pump called a walking beam. Impoundments associated with these wells may or may not be distinct. Figure 5 shows the same area as it appeared in 1979. Three wells appear active. While some ground scarring and impoundments remain, abandoned well locations have become indiscernible as roads and well sites have revegetated.

Wells not active or producing during any of the years of photographs may not exhibit as clearly defined signatures as the producing wells; however, some physical evidence may remain.

For this reason, abandoned wells were separated into three categories, depending on the reliability of the apparent photo characteristics. These include: 1) abandoned well; 2) probable abandoned well or dry hole; 3) possible abandoned well or dry hole.

Figure 2. Active oil wells in 1937 located in the Moore study area.

Figure 3. A 1981 aerial photograph of the same portion of the Moore study area at a slightly smaller scale.

Figure 4. Active oil wells in 1951 located in the Arcadia study area.

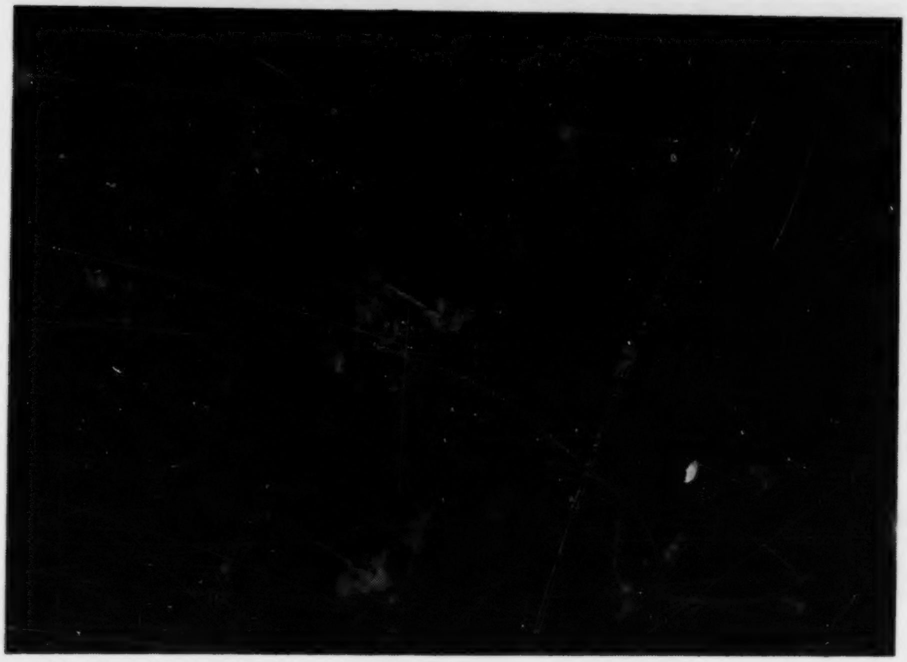


Figure 5. Active oil wells in 1979 located in the Arcadia study area.

Abandoned Well is a classification for those wells that were once producing (as evidenced by oil stains, derricks, walking beams, etc.) and are now inactive.

Probable Abandoned Well or Dry Hole includes well sites for which a definite signature exists (e.g., derrick foundation or walking beam foundation) although the site was never active during any year of photography.

Possible Abandoned Well or Dry Hole is a category for sites that do not exhibit enough photocharacteristics for a positive signature identification; however, they do have enough to suggest past extraction activity (i.e., ground scars, remnants of pits or roads). The location of the suspected wells at these sites is an estimation depending on such factors as terrain and the apparent well spacing pattern.

Figure 6 is a 1951 photograph showing a portion of the Horseshoe Lake area in which possible abandoned wells or dry holes (black arrows) were identified along with active wells (white arrows). By 1979 (Figure 7) the landscape had changed and only a few ground scars remained as evidence of old well sites.

Results of Photo Survey

From the photo analysis a total of 333 abandoned wells and 76 active wells were located within the 24-section study area. A listing of wells by type and section is shown in Tables 3 through 6.

Figure 8 depicts the process that was used to compare the results from the photo analysis to the records search. Table 7 summarizes the types of wells not identified during the initial photo analysis.

Adding up the number of well matches, the number of wells identified from only the photographs, and the number of wells that exhibit some possible photographic evidence but were missed during the initial analysis (left-hand columns of Figure 8) gives a total well count of 442. This number was used as a control to measure the accuracy of the initial photographic analysis. There were 403 of these wells identified from the aerial photographs, thus providing a 91 percent accuracy rate for this pilot study. It is possible that additional wells exist that were not identified from the aerial photographs or the records. However, this number is probably very small and would not significantly change the accuracy rate of the photo analysis.

The cost of acquiring and interpreting aerial photographs was $612 per section.

The Magnetic Survey

In the first part of the study conducted by the USGS, ground-based magnetic measurements (Figure 9) were made near a number of known wells in Colorado to develop a mathematical model of the magnetic anomaly from

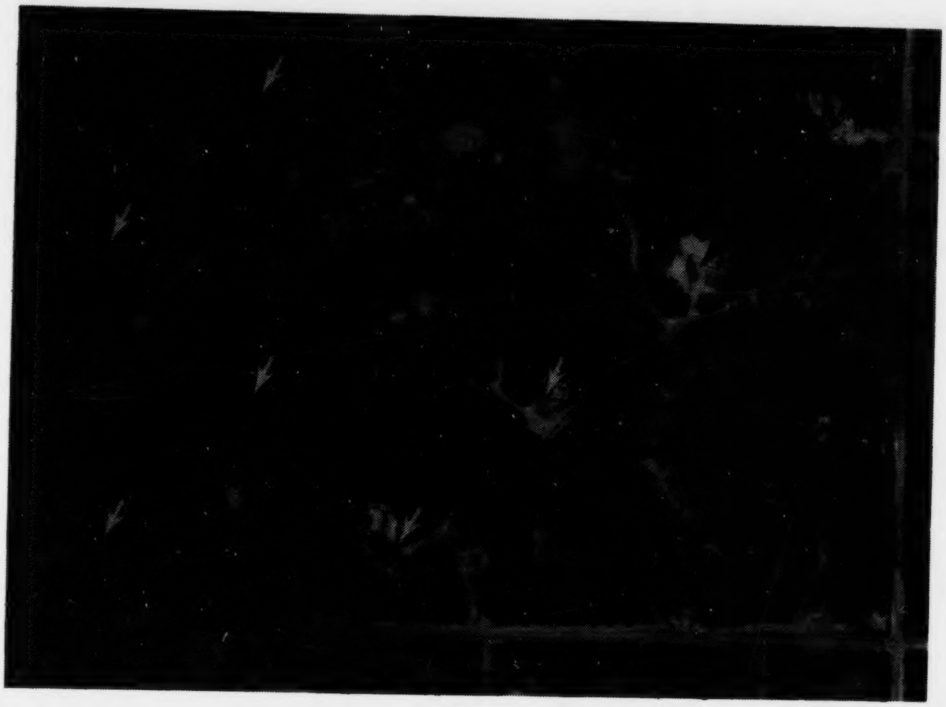


Figure 6. Portion of Horseshoe Lake study area in 1951.

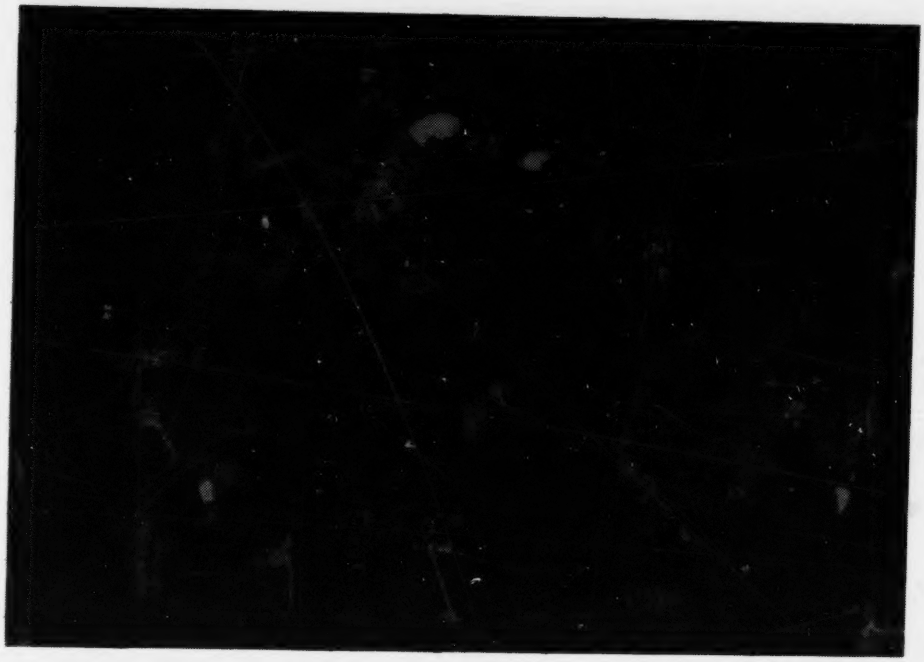


Figure 7. A 1979 aerial photograph of the same portion of the Horseshoe Lake study area.

Table 3. Number of Well Sites Found in Arcadia Study Area

Section	Abandoned	Probable Abandoned or Dry Hole	Possible Abandoned or Dry Hole	Active	Total
3	29	0	3	4	36
10	10	0	5	0	15
11	7	1	3	0	11
14	8	2	5	0	15

Table 4. Number of Well Sites Found in Horseshoe Lake Study Area

Section	Abandoned	Probable Abandoned or Dry Hole	Possible Abandoned or Dry Hole	Active	Total
22	16	0	0	0	16
23	13	0	3	0	16
26	20	0	4	0	24
27	15	0	3	0	18
28	23	1	2	0	26
29	15	0	0	1	16
32	10	0	3	0	13
33	10	0	1	0	11

Table 5. Number of Well Sites Found in Moore Study Area

Section	Abandoned	Probable Abandoned or Dry Hole	Possible Abandoned or Dry Hole	Active	Total
21	8	2	1	9	18*
27	10	7	2	1	20
28	6	2	0	4	11†
29	6	3	5	8	22
32	1	1	4	6	12

*Two abandoned wells that later become injection wells were only counted once.

†An abandoned well/dry hole that later became an injection well was only counted once.

Table 6. Number of Well Sites Found in Oklahoma City Study Area

Section	Abandoned	Probable Abandoned or Dry Hole	Possible Abandoned or Dry Hole	Active	Total
9	9	0	0	6	15
14	9	0	0	5	14
15	11	0	1	5	17
16	12	0	0	4	16
22	12	0	0	4	16
23	8	0	0	8	16
24	4	0	0	11	15

a steel-cased well. Total field anomalies of 1,500 to 6,000 gammas were measured from the wells relative to the earth's magnetic field strength of 55,000 gammas (Figures 10, 11). The mathematical model was used to predict the strength of the magnetic anomaly at various altitudes and distances from the well bore. Figure 12 depicts the size and shape of the magnetic field at a 200-foot altitude determined by the mathematical model for a Colorado test well. (Aircraft flight line spacings of 200 feet would be sufficient in detecting the well assuming a noise level of one gamma in the measurement.) The mathematical model indicated that aircraft flights at 200-foot altitudes and 200- to 300-foot spacings were warranted to evaluate further the use of aeromagnetic surveys for detecting oil and gas wells with steel casing.

Airborne measurements were made in Colorado, prior to measurements in Oklahoma, to validate the modelling results. The mathematical modelling and field measurements in Colorado were used to optimize the aeromagnetic survey for Oklahoma. The model predicted that the magnetic anomaly would decrease in magnitude and increase in size as the aircraft altitude increased. Field data from an overflight of a Colorado test well is plotted in Figure 13 to depict this phenomenon.

Federal Aviation Administration (FAA) regulations restrict the minimum altitude of airplane and helicopter flights. The minimum flight altitudes are lower over rural areas with helicopters having fewer restrictions than airplanes. The USGS used a Fairchild Porter aircraft (Figure 14), modified for short take-offs and landings, in the Oklahoma study. All flights were made in the rural portions of the 24 square miles since magnetometer readings over urban areas would be rendered useless in locating oil and gas wells because of interferences from other man-made objects. The airplane proved to be an effective platform for the magnetometer, even though a helicopter may be better suited for flying at low altitudes, particularly over urban areas. The aircraft proved to be less costly to fly.

Low-altitude, high-resolution aeromagnetic surveys require a radar altimeter, specialized radio navigation systems and compensation techniques for reducing the effect of the aircraft on the recorded magnetic

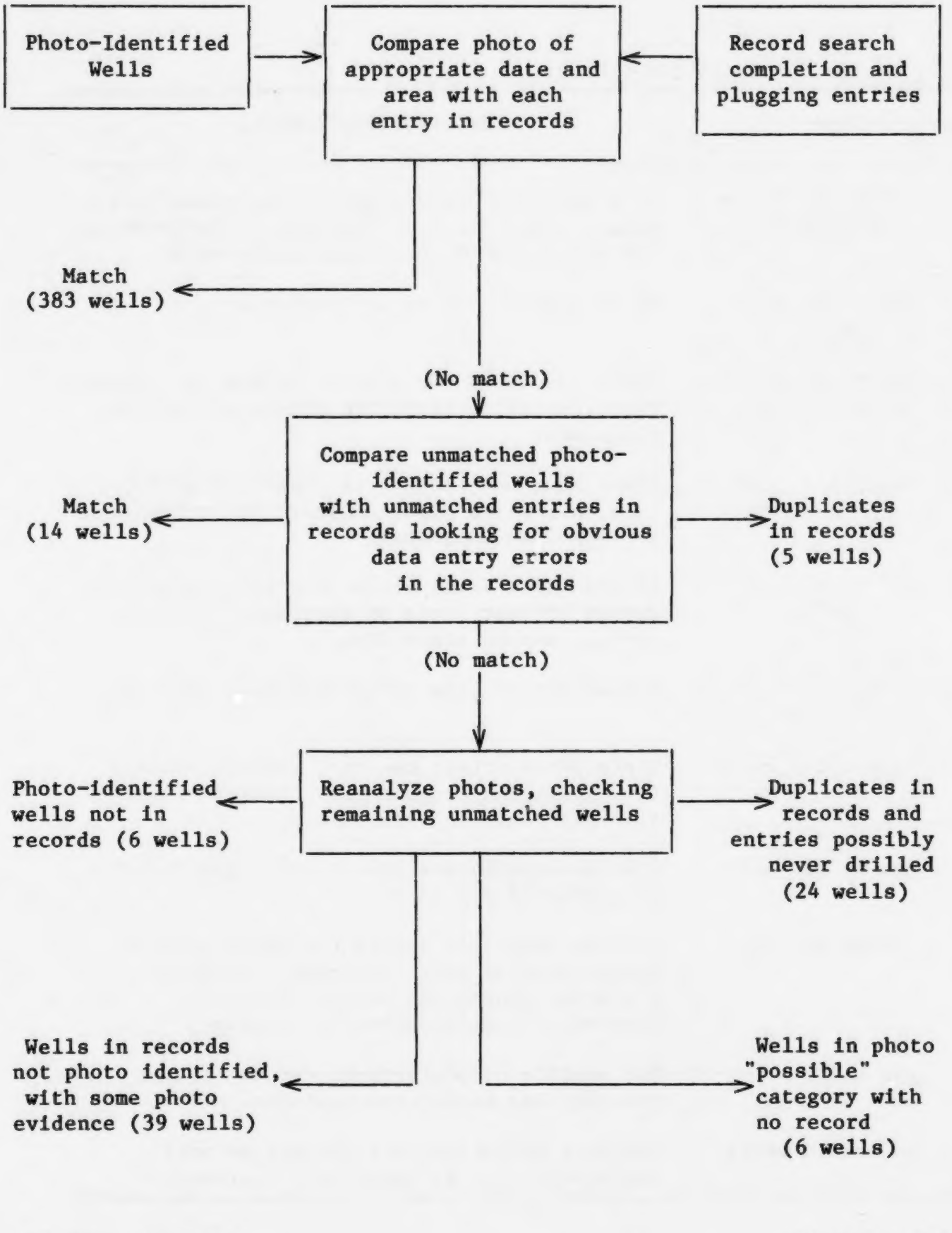


Photo-Identified Wells
Compare photo of appropriate date and area with each entry in records
Record search completion and plugging entries
Match (383 wells)
(No match)
Compare unmatched photo-identified wells with unmatched entries in records looking for obvious data entry errors in the records
Match (14 wells)
Duplicates in records (5 wells)
(No match)
Reanalyze photos, checking remaining unmatched wells
Photo-identified wells not in records (6 wells)
Duplicates in records and entries possibly never drilled (24 wells)
Wells in records not photo identified, with some photo evidence (39 wells)
Wells in photo "possible" category with no record (6 wells)
442 Total # Wells
403/442 = 91%

Figure 8. Comparison Process Used to Determine Accuracy of Photo Survey.

Table 7. Wells Not Photo Identified

Number	Type	Description/Comments
12	early dry holes (1937-1951)	Site may show one feature of signature (i.e., ground scar, trail or remnants of impoundment) but not enough to establish uniqueness.
1	early gas well (1919)	Faint ground scar on early photographs.
2	early oil wells (1947-1950)	Wells abandoned and plugged before '51 photographs, plugging disturbed ground and masked signature.
2	early/mid year	Sites show one feature of signature (structure, oil wells pipeline scar) but not enough to establish uniqueness.
4	mid-year oil wells	Records show wells active for short time only during 12-year lapse of coverage. Land use changes masked signatures.
3	mid-year oil wells	Missed due to tree canopy and busy land use.
7	recent dry holes	Disturbance noted; however, features didn't fit established signatures. Easy to identify with signature refinement.
1	recent dry hole	Only photo evidence is a trail. Questionable existence of dry hole.
1	recent oil well	Records show well active for short time between years of photo coverage. Plugging disturbed ground and masked signature. Incorrect location given in records.
4	gas wells	Not visible on photographs due to lack of recognizable associated features.
2	injection wells	Analysts unfamiliar with injection well configuration. No associated features.

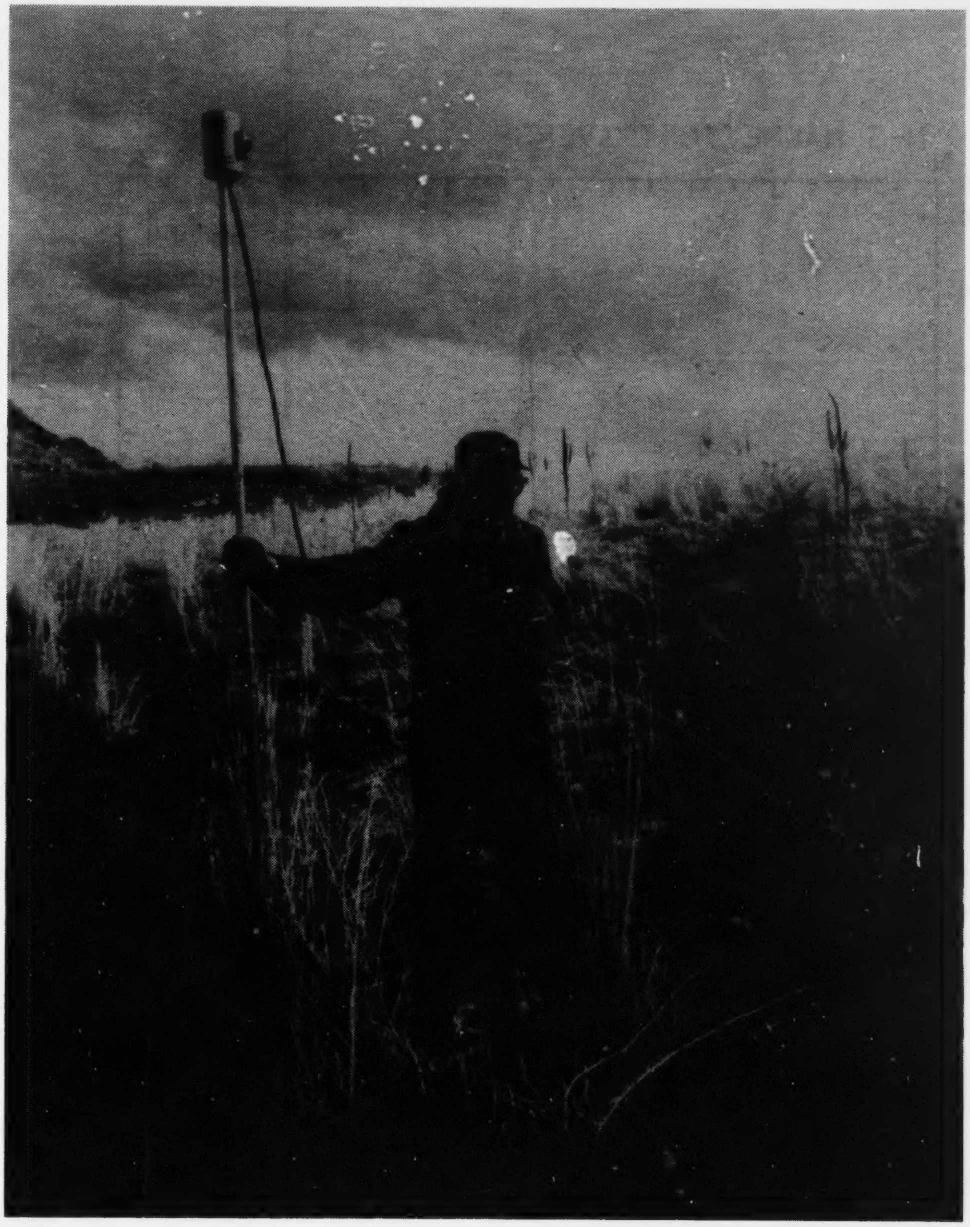


Figure 9. Portable proton magnetometer in use in Colorado.

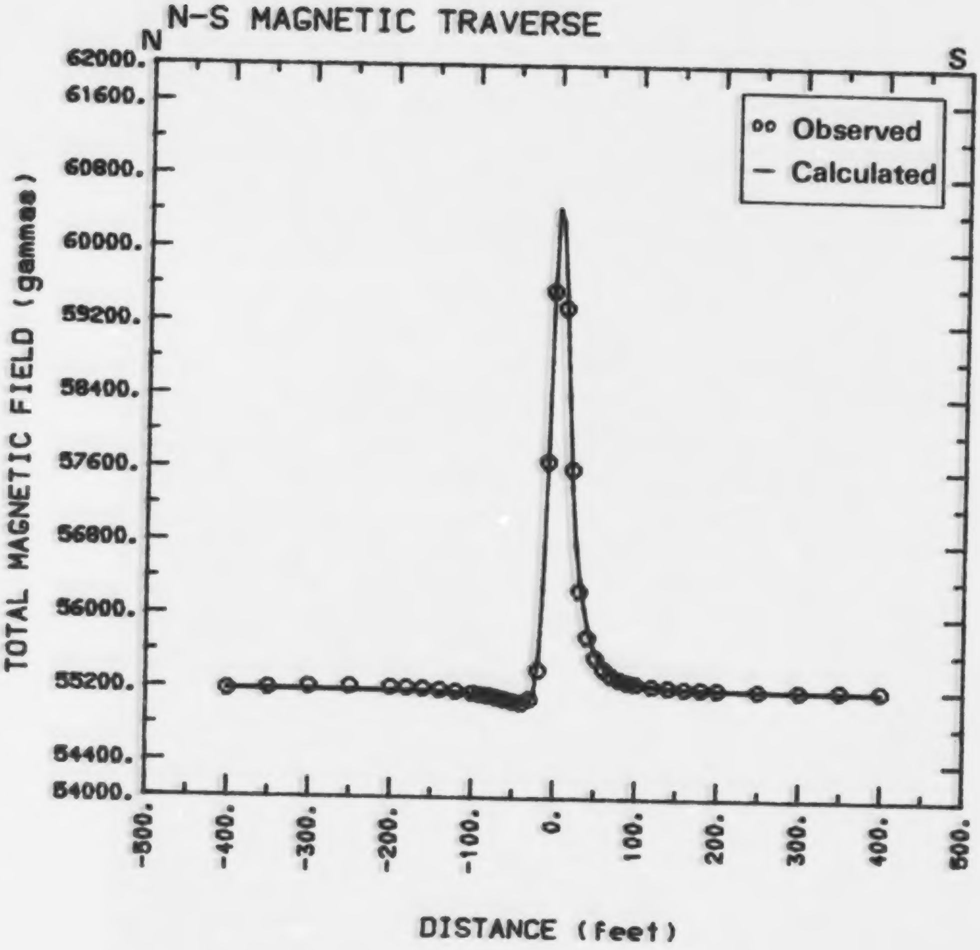


Figure 10. North-south ground magnetic profile over well no. 4.

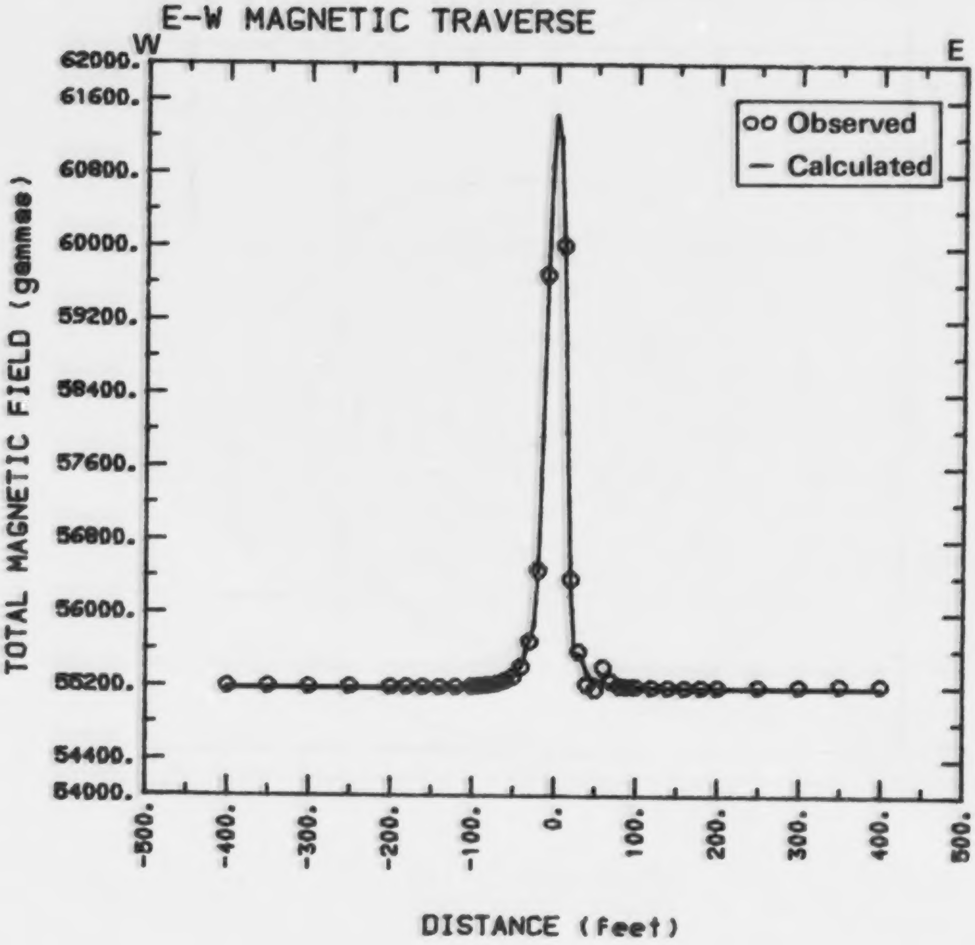


Figure 11. East-west ground magnetic profile over well no. 4.

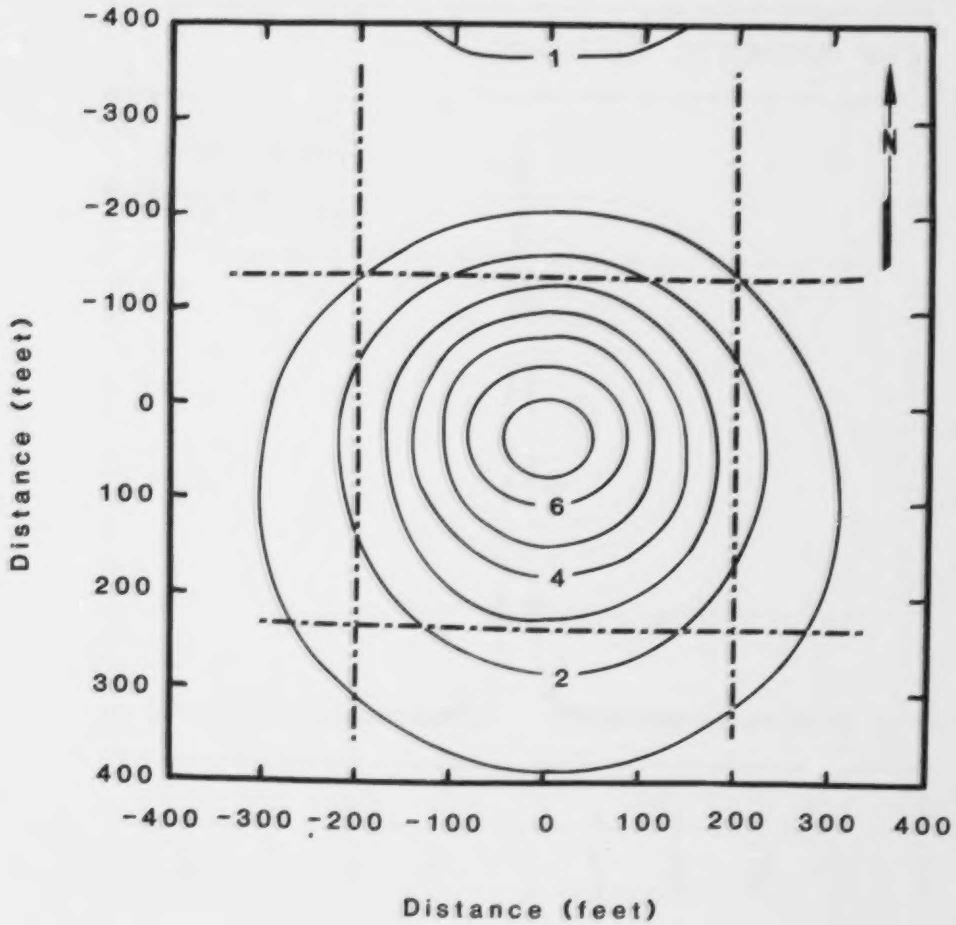


Figure 12. Calculated isopleth map of total intensity at a height of 200 feet above a well. The lines show spacing of north-south or east-west flight lines necessary to measure a two-gamma anomaly in the worst case.

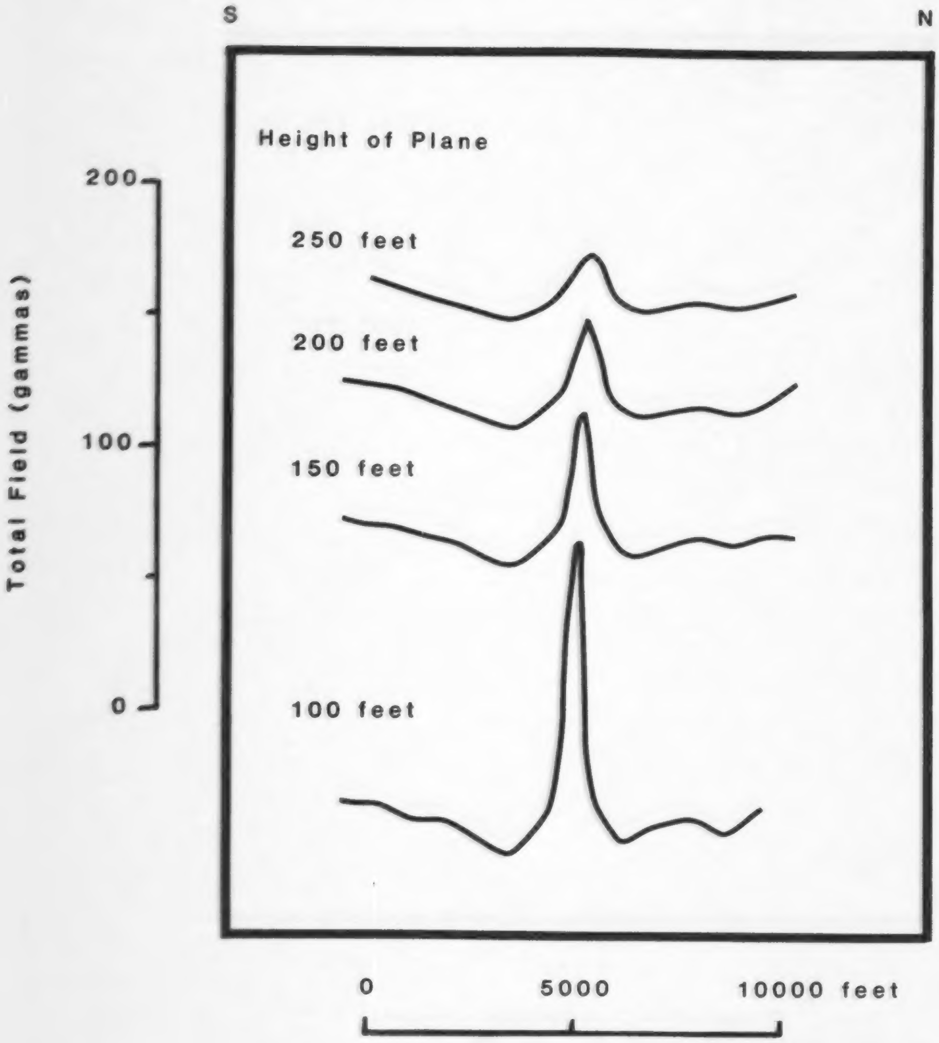


Figure 13. Aeromagnetic profiles for different aircraft heights over well no. 4, Piney Creek, Colorado area.

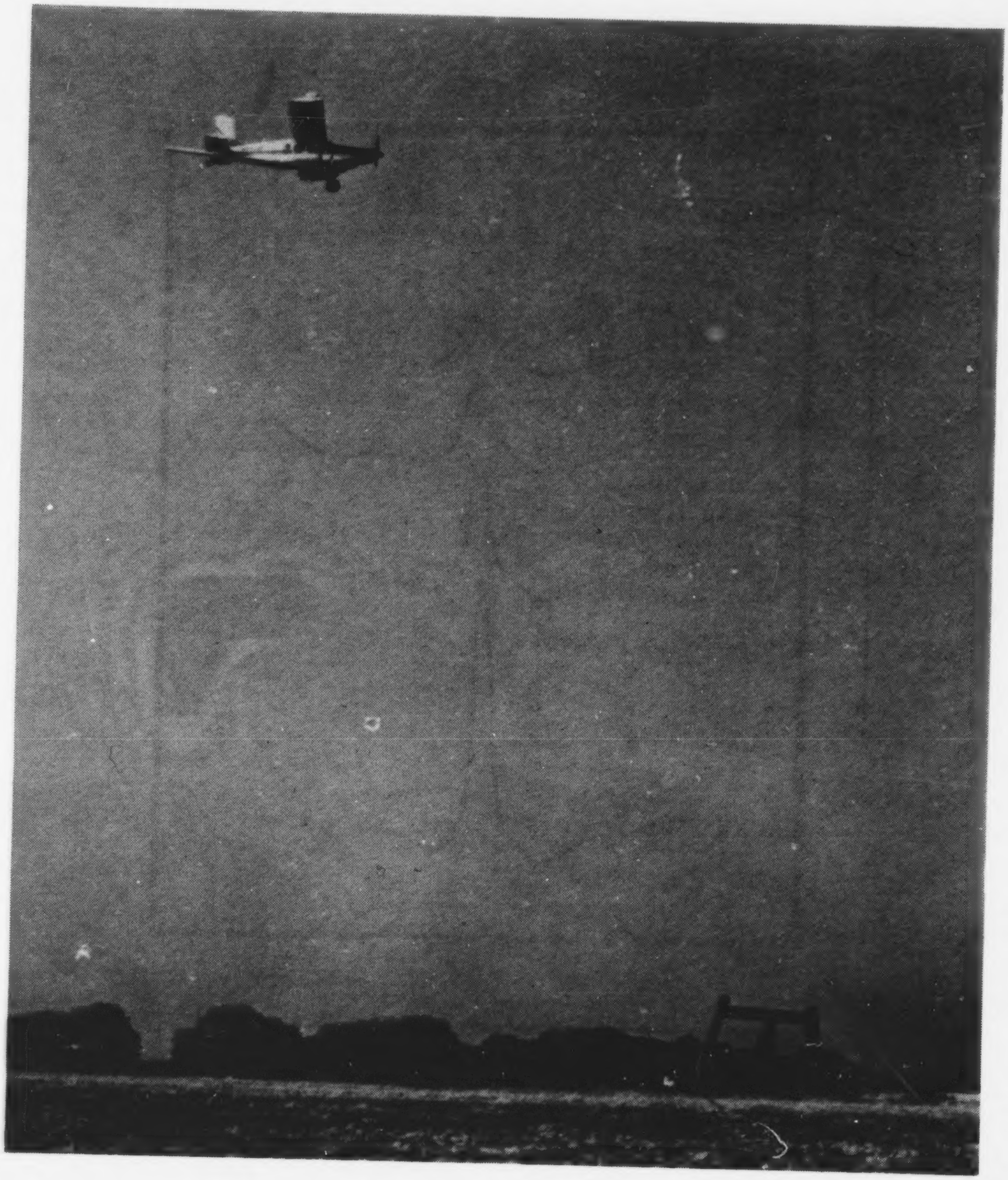


Figure 14. Fairchild Porter aircraft used in this study.

field. The USGS used small, lightweight, battery-powered transponders at surveyed points to establish the position of the aircraft to a relative accuracy of ±16 feet. An indicator in the cockpit permitted the pilot to fly an imaginary flight line while the actual position of the aircraft was recorded on magnetic tape. The altitude of the aircraft was measured and recorded on magnetic tape (Figure 15) and the data were used to compensate for the changing magnetic field created by the motion of the aircraft in the earth's field. Non-ferrous metals and electrical coils were installed in the aircraft at critical points to reduce the influence of the aircraft on the proton-precision magnetometer carried in a tail "stinger."

An example of the reduced field data from an aeromagnetic survey of the Arcadia test area is presented in Figure 16. An examination of aerial photographs and some ground-based magnetometer measurements was required to resolve questions between well locations identified by the USGS and well locations identified by either EPIC or the Environmental and Ground Water Institute. Between 95 to 98 percent of the probable number of oil and gas wells were located in the sections surveyed by the USGS; the exact figure varied from one section to another. When wells were clustered together, the magnetic anomaly detected from the air could not be resolved to locate the individual wells; however, ground-based measurements were successful in discriminating individual wells from geologic and man-made features.

The cost of performing an aeromagnetic survey has been estimated to be between $1,000 to $2,000 per square mile depending on the total size of the area surveyed. While the use of magnetometers proved to be the most expensive and complicated method for locating abandoned wells, the aeromagnetic survey together with the ground-based magnetometer measurements found the greatest number of wells and with the greatest accuracy in locating an individual cased well. Most wells containing approximately 200 feet or more of 8-5/8-inch diameter or larger surface casing can be detected by airborne measurements.

Summary and Conclusions

Three methods were used to locate abandoned oil and gas wells in Oklahoma with varying degrees of cost and success. As each method was used in succession to survey the study areas, the level of confidence that all abandoned wells had been located increased, but each method also raised the total cost of the investigation. The records search provided information on well construction which the other two techniques cannot supply; therefore, it is likely that a records search will always be required to assess the pollution potential from abandoned wells. Unfortunately, the information contained in the records on both well location and construction may not be complete or accurate. Additional location techniques are desirable to supplement the data.

Historical aerial photographs are particularly valuable for those periods when records are not complete or accurate. This is particularly true for the period from the 1930's through 1950's during which improved, wide-spread photographic coverage became available and accurate records

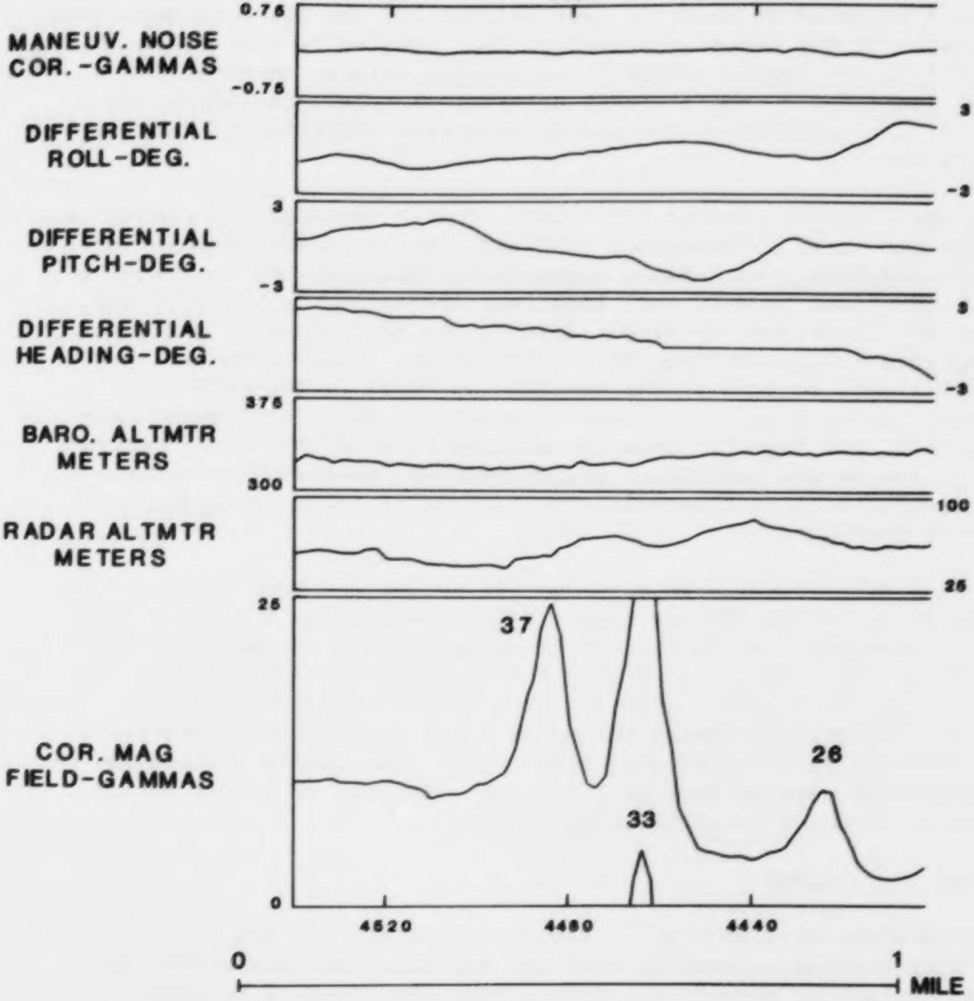
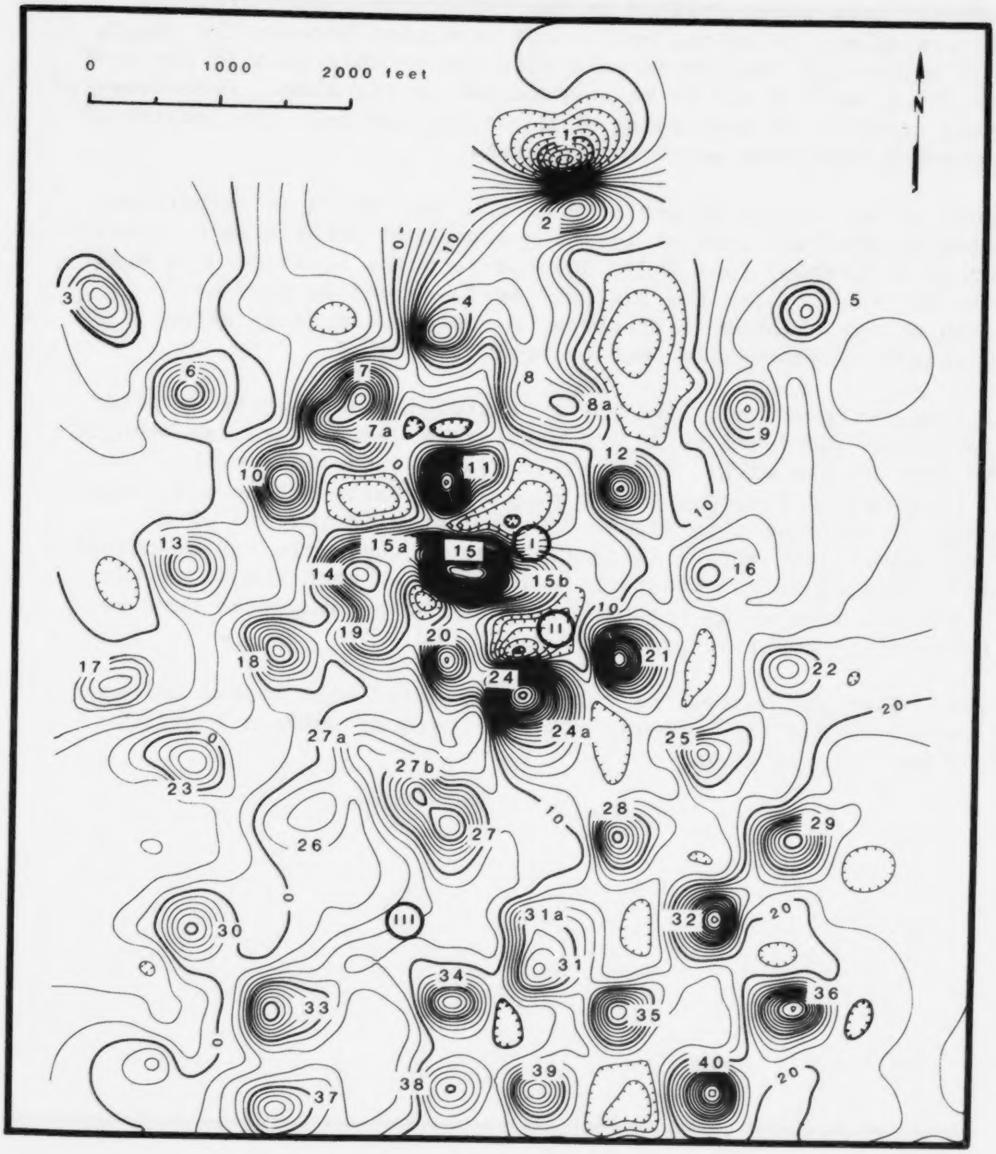


Figure 15. Airborne profile data from Arcadia area. The numbers at top and bottom are identification numbers associated with each reading, and the numbered anomalies correspond with those on Figure 16.


0
1000
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N
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31a
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38
39
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I
II
III
0
10
20

Figure 16. Total magnetic intensity isopleth map for part of Arcadia area.

were not often required. In areas where rapid land use changes have occurred, it can be difficult to locate abandoned wells when the length of drilling time at a site was short in relation to the period of time between photos. Even wells drilled in the recent past, when frequent photographs are likely to exist, can escape detection because the length of time that modern-day rigs spend on a site can be less than in the past when the drilling derrick had to be constructed at the site. Fortunately, the increased emphasis on developing good records has made the problem of locating recently abandoned wells much easier.

The cost of an historical aerial photographic search is comparable to the estimated $50/well cost of locating abandoned wells from a records search (Arthur D. Little, Inc. 1979) and the accuracy in locating a well is apt to be better. (Using the $50/well estimate and an average 17 wells in each of the Oklahoma test areas, a rough estimate in using records to search a square-mile section would be $850.)

The aeromagnetic method, like the photographic method, can be readily used to locate abandoned wells for many areas where there has been no surface evidence of the well. Large areas can be surveyed rapidly from the air without need for access to the property. While the method allows a well casing to be located to within 3 to 6 feet with the aid of a ground-magnetometer, the method is costly. An aeromagnetic survey requires more sophisticated equipment and technical expertise than the other two methods. However, more wells were detected by the aeromagnetic surveys than by the initial photointerpretation.

For any survey the method, or methods, selected will depend on the available resources and the potential threat posed by unknown locations of abandoned wells in an area.

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ABANDONED WATER WELLS IN SOUTHEASTERN MINNESOTA

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Abstract

There is much concern in southeastern Minnesota about the degradation of the quality of the ground water. Both near-surface and deep aquifers exhibit signs of contamination. Several factors have contributed to this degradation, including karst features such as fractured and jointed bedrock, little to no soil cover, and sinkholes. In addition, improperly abandoned water wells and poorly constructed water wells contribute to the problem. These factors all allow the direct communication of surface and ground water. Numerous studies have shown that these factors have negative effects on ground water quality.

The Minnesota Department of Natural Resources (DNR) began a study to determine the extent of the problem of abandoned wells on DNR land within southeastern Minnesota. A total of 113 abandoned water wells were found in 8 counties. Fifteen accessible wells not obstructed with drop pipes or pumps were geophysically logged. The geophysical logs and additional data from other abandoned wells revealed that such wells may act as conduits for surficially-derived contaminants and that approximately half of them are uncased through two or more aquifers, allowing for the exchange of water and water-borne contaminants between aquifers. These wells are unstable and are prone to collapse. Approximately 70% of the located wells had collapsed, many of them in the past few years; one of the logged wells collapsed between visits of the logging crew. After the collapse of a multiaquifer well, it may not be possible to achieve a water-tight seal without redrilling the well. Obviously, grouting before collapse is more cost-effective than redrilling. Whichever method is employed, isolation of the aquifers is imperative regardless of cost.

Introduction

Southeastern Minnesota is an agricultural region with numerous farms, small towns, and a few moderately-sized cities. The area is dissected by several streams and rivers that flow east from the gently rolling uplands toward the Mississippi River. Elevation varies from 625 feet to 1200 feet above sea level. The eight counties of southeastern Minnesota are Dodge, Fillmore, Goodhue, Houston, Mower, Olmstead, Wabasha, and Winona. They are of prime concern because they have near-surface carbonate bedrock, forming an active karst terrain abundant in sinkholes, caves, sinking streams, and springs. Ground water quality in this area of Minnesota has become a major concern as more of this resource becomes contaminated. Since residents obtain most of their water from wells, a greater understanding of the ground water system and additional measures to protect ground water quality are necessary.

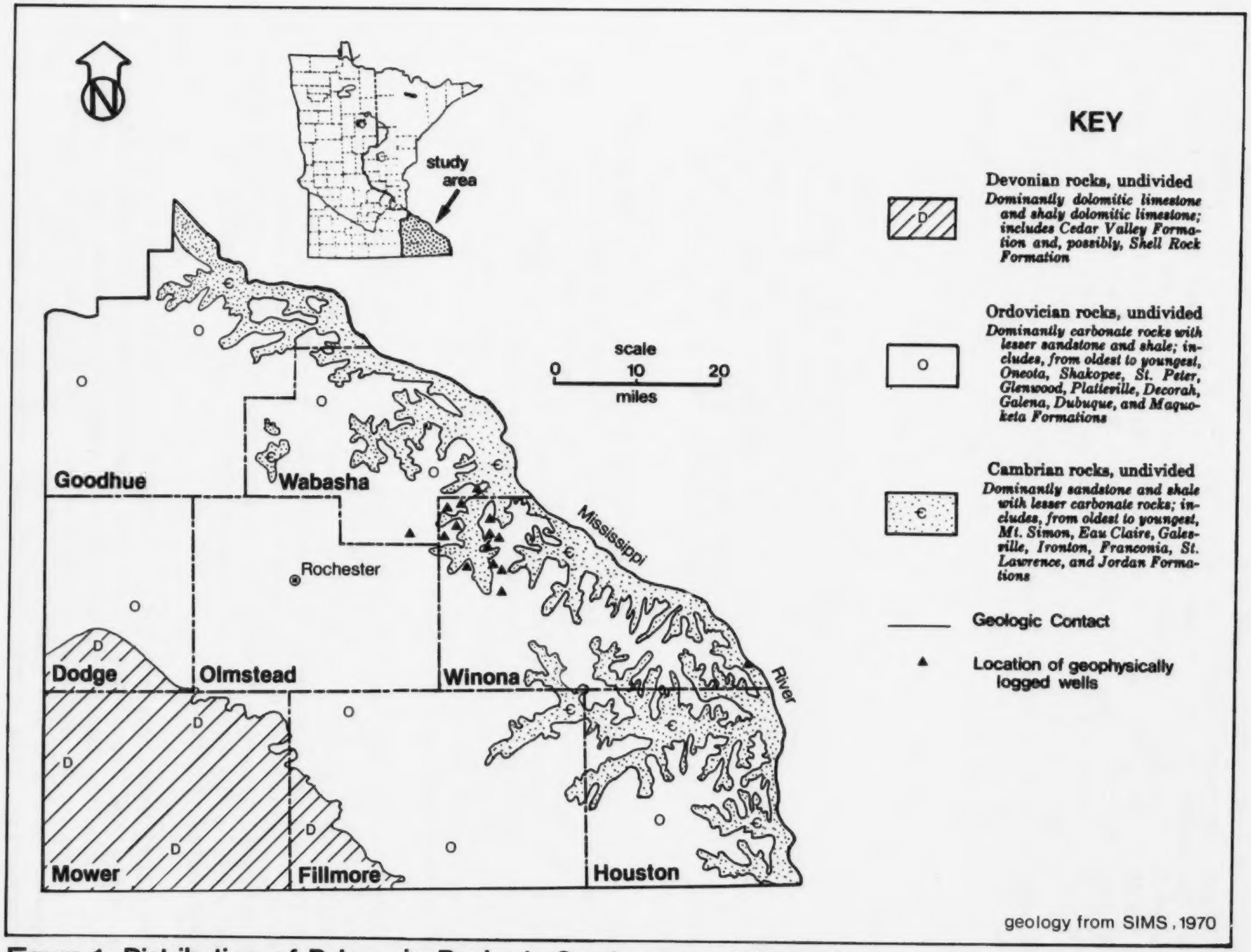
Hydrogeology

Southeastern Minnesota is underlain by a thick Paleozoic section of alternating layers of sandstone, shale, and carbonates. Most of the bedrock units present are comprised of Cambrian, Ordovician, and Devonian Age material (see Figure 1). The different stratigraphic units are depicted in Figure 2. The subsurface geologic conditions are fairly uniform and flat laying, with a regional dip to the west-southwest of less than one degree (Thiel, 1944).

Ground water is abundant in the Paleozoic sandstones and carbonates; water for domestic and municipal use is drawn from five bedrock aquifers. These include an upper carbonate aquifer (usually the Galena), the St. Peter, Prairie du Chien-Jordan, Franconia-Ironton-Galesville, and Mount Simon aquifers (see Figure 3). The use of these various aquifers is dependent upon water quality and quantity. Bedrock units close to the surface may contain elevated levels of nitrate or they may be dewatered if located on a ridge system (Kanivetsky, 1984).

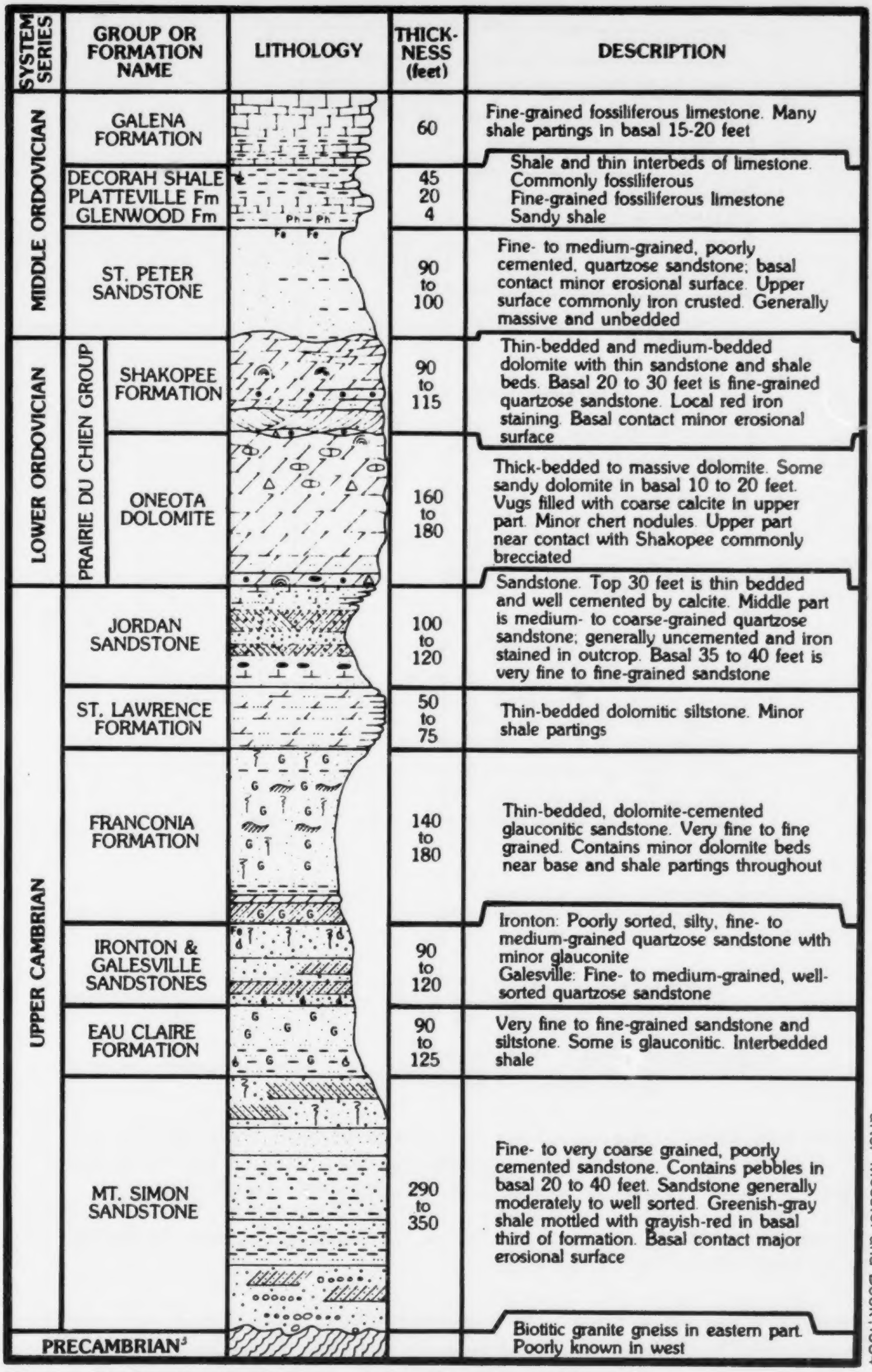
The aquifers are separated by confining layers having low permeabilities (see Figure 3). The resulting head difference of the aquifers can be quite large, on the order of 180 feet between the Prairie du Chien-Jordan aquifer and the Franconia-Ironton-Galesville aquifer in Winona County (Olsen, 1984). It is this head difference that allows water to percolate downward from one aquifer to another when the confining layer has been penetrated, as in the case of open boreholes, abandoned wells, or poorly constructed wells.

The regional hydrology exhibits a subsurface flow to the northeast, generally toward the Mississippi River and its tributaries (Broussard *et al.*, 1975). Since the hydraulic head of each aquifer decreases with depth, water moves downward in addition to laterally. Research done by Alexander *et al.* (1984), Dalgleish and Alexander (1984), and Mohring (1983) have shown the subsurface flow within the carbonate units to be complex and variable.


study area
scale
0 10 20
miles
Goodhue
Wabasha
Rochester
Dodge
Olmstead
Winona
Mower
Fillmore
Houston
Mississippi
River
KEY
Devonian rocks, undivided
Dominantly dolomitic limestone and shaly dolomitic limestone; includes Cedar Valley Formation and, possibly, Shell Rock Formation
Ordovician rocks, undivided
Dominantly carbonate rocks with lesser sandstone and shale; includes, from oldest to youngest, Oneota, Shakopee, St. Peter, Glenwood, Platteville, Decorah, Galena, Dubuque, and Maquoketa Formations
Cambrian rocks, undivided
Dominantly sandstone and shale with lesser carbonate rocks; includes, from oldest to youngest, Mt. Simon, Eau Claire, Galesville, Ironton, Franconia, St. Lawrence, and Jordan Formations
Geologic Contact
Location of geophysically logged wells

geology from SIMS, 1970

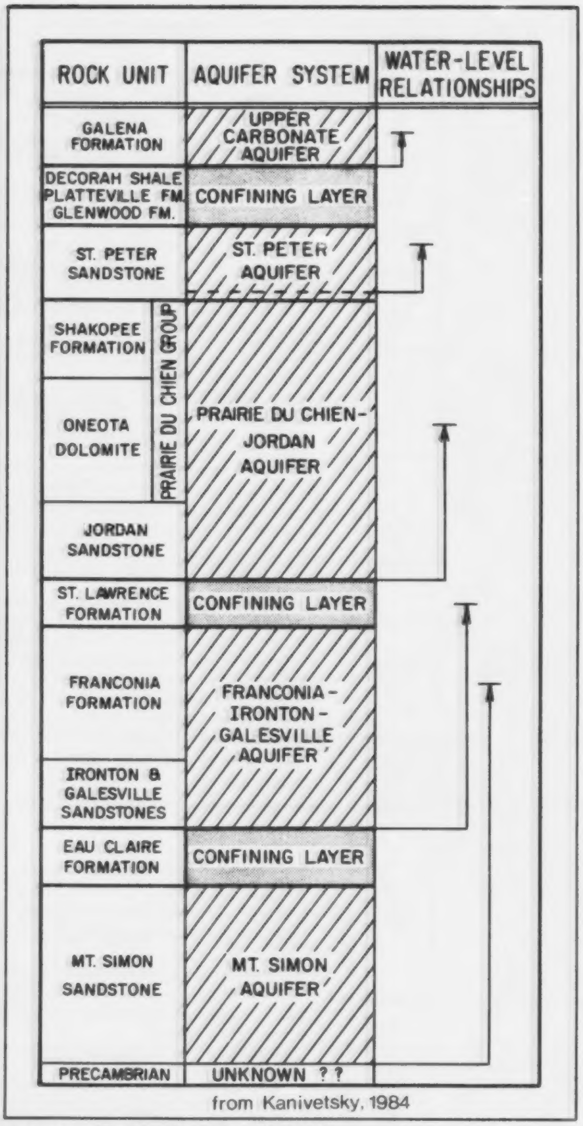
Figure 1. Distribution of Paleozoic Rocks in Southeastern Minnesota.



SYSTEM SERIES	GROUP OR FORMATION NAME		LITHOLOGY	THICK-NESS (feet)	DESCRIPTION
MIDDLE ORDOVICIAN	GALENA FORMATION			60	Fine-grained fossiliferous limestone. Many shale partings in basal 15-20 feet
	DECORAH SHALE			45	Shale and thin interbeds of limestone. Commonly fossiliferous
	PLATTEVILLE Fm			20	Fine-grained fossiliferous limestone
	GLENWOOD Fm			4	Sandy shale
	ST. PETER SANDSTONE			90 to 100	Fine- to medium-grained, poorly cemented, quartzose sandstone; basal contact minor erosional surface. Upper surface commonly iron crusted. Generally massive and unbedded
LOWER ORDOVICIAN	PRAIRIE DU CHIEN GROUP	SHAKOPEE FORMATION		90 to 115	Thin-bedded and medium-bedded dolomite with thin sandstone and shale beds. Basal 20 to 30 feet is fine-grained quartzose sandstone. Local red iron staining. Basal contact minor erosional surface
		ONEOTA DOLOMITE		160 to 180	Thick-bedded to massive dolomite. Some sandy dolomite in basal 10 to 20 feet. Vugs filled with coarse calcite in upper part. Minor chert nodules. Upper part near contact with Shakopee commonly brecciated
UPPER CAMBRIAN	JORDAN SANDSTONE			100 to 120	Sandstone. Top 30 feet is thin bedded and well cemented by calcite. Middle part is medium- to coarse-grained quartzose sandstone; generally uncemented and iron stained in outcrop. Basal 35 to 40 feet is very fine to fine-grained sandstone
	ST. LAWRENCE FORMATION			50 to 75	Thin-bedded dolomitic siltstone. Minor shale partings
	FRANCONIA FORMATION			140 to 180	Thin-bedded, dolomite-cemented glauconitic sandstone. Very fine to fine grained. Contains minor dolomite beds near base and shale partings throughout
	IRONTON & GALESVILLE SANDSTONES			90 to 120	Ironton: Poorly sorted, silty, fine- to medium-grained quartzose sandstone with minor glauconite Galesville: Fine- to medium-grained, well-sorted quartzose sandstone
	EAU CLAIRE FORMATION			90 to 125	Very fine to fine-grained sandstone and siltstone. Some is glauconitic. Interbedded shale
	MT. SIMON SANDSTONE			290 to 350	Fine- to very coarse grained, poorly cemented sandstone. Contains pebbles in basal 20 to 40 feet. Sandstone generally moderately to well sorted. Greenish-gray shale mottled with grayish-red in basal third of formation. Basal contact major erosional surface
PRECAMBRIAN[3]					Biotitic granite gneiss in eastern part. Poorly known in west

after Mossler and Book, 1984

Figure 2. Geologic Column of Southeastern Minnesota


ROCK UNIT
AQUIFER SYSTEM
WATER-LEVEL RELATIONSHIPS
GALENA FORMATION
UPPER CARBONATE AQUIFER
DECORAH SHALE
PLATTEVILLE FM.
GLENWOOD FM.
CONFINING LAYER
ST. PETER SANDSTONE
ST. PETER AQUIFER
SHAKOPEE FORMATION
ONEOTA DOLOMITE
PRAIRIE DU CHIEN GROUP
PRAIRIE DU CHIEN-JORDAN AQUIFER
JORDAN SANDSTONE
ST. LAWRENCE FORMATION
CONFINING LAYER
FRANCONIA FORMATION
IRONTON & GALESVILLE SANDSTONES
FRANCONIA-IRONTON-GALESVILLE AQUIFER
EAU CLAIRE FORMATION
CONFINING LAYER
MT. SIMON SANDSTONE
MT. SIMON AQUIFER
PRECAMBRIAN
UNKNOWN ? ?

from Kanivetsky, 1984

Figure 3. Bedrock Aquifers of Southeastern Minnesota

Recharge occurs from precipitation; discharge occurs from springs, seeps, and streams in the valleys. Alexander et al. (1984) have documented the flow of ground water in karst areas from upland recharge areas to private domestic wells. Book and Alexander (1984) have shown that effluent from a small community sewage treatment lagoon entered an upland sinkhole to later emerge from three local springs.

Deeper aquifers are of greater use in the eastern portion of the study area relative to the western. This is due to the dewatering of the upper aquifers in the incised valleys, which cut into the flat-lying bedrock units on their way to the Mississippi River. The deepest aquifer, the Mt.

Simon, is present everywhere in southeastern Minnesota, but it is used infrequently because of prohibitive drilling costs. However, bedrock units near the surface may contain elevated levels of nitrate (Olsen, 1984), and thus make use of deeper aquifers necessary.

Ground Water Quality

The quality of ground water within southeastern Minnesota has severely deteriorated in recent years. Deeper aquifers are exhibiting signs of degradation. Some municipal wells, properly constructed into the Jordan sandstone have nitrate levels exceeding the 10 ppm standard established by the National Primary Drinking Water Regulations (Minnesota Department of Health, 1976).

Degradation of ground water quality within this area is caused by several factors. The fractured nature of the carbonate bedrock, little to no soil cover, sinkholes, abandoned water wells, and poorly constructed wells all may act as pathways for contaminants; ground water enters the karst aquifer without receiving sufficient natural filtration.

Karst terrain is especially sensitive to ground water contamination. The creviced and fractured nature of limestones and dolomites (e.g. Oneota Dolomite) allows contaminants to travel great distances. Various studies by Alexander et al. (1984) and Milske and Alexander (1984) have shown that ground water movement within karst aquifers occurs at much greater velocities than in sandstone aquifers, which can be on the order of miles per week versus feet per year. These higher velocities within carbonate aquifers mean that contaminants from a surface source may reach wells located miles away in a short period of time (Kanivetsky, 1984; Milske and Alexander, 1984).

The thin cover of unconsolidated material is usually less than 50 feet, and can commonly be less than 30 feet (Kanivetsky, 1984; Olsen and Mossler, 1982). Surface water receives almost no natural attenuation of pollutants before becoming a part of the ground water. Sinkholes, abandoned wells, and poorly constructed wells offer direct communication between surface and ground water.

Previous Work

Residents and local officials have been concerned about declining water quality in the karst area of Minnesota for many years. A Minnesota Department of Health (MDH) report (1976) to the Minnesota Legislative Commission on Minnesota Resources reported that 4 counties in southeastern Minnesota have the most serious water contamination problems. Eighty five percent (85%) of the water wells surveyed in Fillmore, Wabasha, and Winona Counties showed some evidence of contamination, wells in which nitrate-nitrogen concentrations were found to be greater than 1.0 ppm. Research of rural water supplies by Mierau (1971a, 1971b, and 1974) found the main contaminants to be coliform bacteria and nitrate (see Table 1). Mierau suggested that much of the contamination could be prevented by upgrading water supply and holding systems.

Table 1. Water Quality Findings of 3 Counties of Southeastern Minnesota

	Fillmore	*Wabasha*	*Winona*
Wells > 1.0 ppm nitrate nitrogen	95%	87%	85%
Wells > 10 ppm	47%	29%	13%
Range of values (ppm)	1.0 to 30	1.0 to 42	1.0 to 46
presence of coliform bacteria	31%	31%	40%

from Mireau, 1971 and 1974

Studies have also been done in the karst of the neighboring states of Wisconsin and Iowa. Sherrill (1978) considered the effects of casing length and grouting techniques on water quality in Door County, Wisconsin. He concluded that proper well construction decreases the possibility of a well yielding contaminated water. In a study of ground water quality in 2 counties of northeastern Iowa, Libra et al. (1984) found that wells drilled in shallow aquifers exhibited significant concentrations of surficially-derived contaminants, all over 22 ppm. They also found that 70-80% of the wells sampled had detectable levels of pesticides. The ground water quality of the near surface bedrock aquifers in northeastern Iowa, "...is clearly being degraded by the infiltration of surface-applied chemicals."

In addition to protecting water quality, the MDH (1976) also considered the problem of abandoned water wells in southeastern Minnesota. They surveyed 5 townships in Fillmore, Houston, Winona, and Wabasha Counties. Their findings suggest that the number of abandoned wells in that portion of Minnesota is approximately equal to the active wells, which number over 11,000.

Some of these abandoned wells have multiaquifer construction, a situation that arises when an uncased borehole penetrates 2 or more aquifers. This type of well allows for the rapid vertical movement of water and contaminants between aquifers. Hult (1979) considered this situation in Fillmore County and concluded that large flows can occur through such wells for extended periods. They are, however, "...mechanically unstable and flow through them tends to decrease with time in response to filling of the borehole and clogging of aquifer pores by sediment."

Since many of the abandoned water wells were discovered to be located on land administered by the Minnesota Department of Natural Resources (DNR), a study was undertaken to determine the locations and histories of the old wells. The methods used to locate the wells have been reported earlier (Blomquist, 1984). The comparative use of old and new topographic maps in concert with old aerial photographs provided most of the information in identifying sites that might contain abandoned water wells. These sites were field checked during 1983-1984 and this information is summarized by Blomquist and Rongitsch (1984).

The Wells

A total of 113 abandoned water wells were found on DNR owned land in the eight counties of southeastern Minnesota (Blomquist and Rongitsch, 1984). The majority of the wells were found on former farmsteads purchased by the DNR for integration into its system of parks, forestry, and wildlife management areas. During the past 50 years, hundreds of such sites have been purchased in southeastern Minnesota, amounting to a total land area of over 68,000 acres. A few structures remain on these sites, but in most cases the buildings were razed and the water wells left unattended.

Topographically, 82 (73%) of the wells were located on ridges and 31 (28%) in valleys. Those located on ridge tops were primarily drilled wells, 300 to 600 feet in depth, while those in the valleys were driven or sandpoint wells, usually no deeper than 30 feet. However, the sandpoints are of less concern because they are shallow, because they do not penetrate bedrock, and because most of them were destroyed at the same time as the surrounding buildings.

Many of the wells reflect the water well construction practices of the 1920's and 30's. Approximately 70% of the drilled wells had casings that extended beyond the unconsolidated layer. Several wells did not have regular steel casing, but rather a material that can best be described as "stove-pipe" casing. This material readily rusts out and thus would allow surface water to flow into the well.

The wells were found in many different conditions of disrepair. Eight wells were covered with water tight seals, 60 had the hardware removed and were left open, and 45 were simply left as is, with no action taken before abandonment. This last category could be referred to as "walk-aways".

Those wells which were free of obstructions (drop pipe, pumps, etc.) and accessible to vehicles were geophysically logged. Downhole geophysical instruments were lowered into them to obtain additional information about the geology, hydrogeology, and well construction. A total of 15 wells were logged in this manner by the Minnesota Geological Survey (MGS).

Table 2 lists several characteristics of these wells; the information is based upon caliper, gamma, and electrical resistivity logs. Columns 1 through 4 contain the information from the geophysical logs: depth, static water level, casing diameter and depth, and bottom rock unit. Columns 5, 6 and 7 contain inferences based upon the interpretation of the logs. The most probable aquifer(s) in each well was deduced from the construction of the well and the known local hydrogeology in answer to the question, "What aquifer could best supply the water needs for this site?" Multiaquifer status was assigned when the borehole was found to be uncased in more than one aquifer. Whether or not a borehole had collapsed was calculated by considering the most probable aquifer in conjunction with the present measured depth and water level. Table 2 shows that 10 of the 15 logged wells (67%) have collapsed since construction and 7 of the 15 (47%) were probably multiaquifer wells. The 15 logged wells represent 13% of the total number of abandoned wells found in this study.

Table 2. Characteristics of Geophysically Logged Wells

Well Name	*Depth (ft)*	*Static Water Level (ft)*	*Casing*	*Bottom Unit*	*Probable Aquifer(s)* [1,2]	*Multiaquifer*[1]	*Collapsed*[1]
Callan	281	D	4½" > 281′	Prairie du Chien	F	NO	YES
Foreman	492	386	6" to 24′	Ironton	P-J-F-I	YES	NO
Kraemer	304	286	4" > 297′	St. Lawrence	F	NO	YES
Krenzke	280	D	4½" > 261′	Jordan	P-J-F	YES	YES
A. Kronebusch	420	310	4" to 320′	Franconia	F	NO	NO
P. Kronebusch	267	D	4" > 254′	Jordan	P-J-F	YES	YES
Luther Haven So.	205	62	4" to 36′	Mt. Simon	G-M	YES	NO
Melcher	489	320	4½" to 322′	Franconia	F	NO	NO
Putnam	198	62	4" to 179′	Eau Claire	M	NO	YES
A. Schmidt	349	D	6" to 316′	St. Lawrence	J-F	YES	YES
B. Schmidt	283	?	6" to 36′	Prairie du Chien	P-J	NO	YES
Siebenaler	302	301	6" to 200′	Ironton	J-F-I	YES	YES
Steffen	275	151	4½" > 60′	Jordan	F	?	YES
W. Wischow	485	355	6" to 82′	Franconia	P-J-F	YES	NO
Zastrow	280	D	4½" > 280′	Jordan	F	?	YES

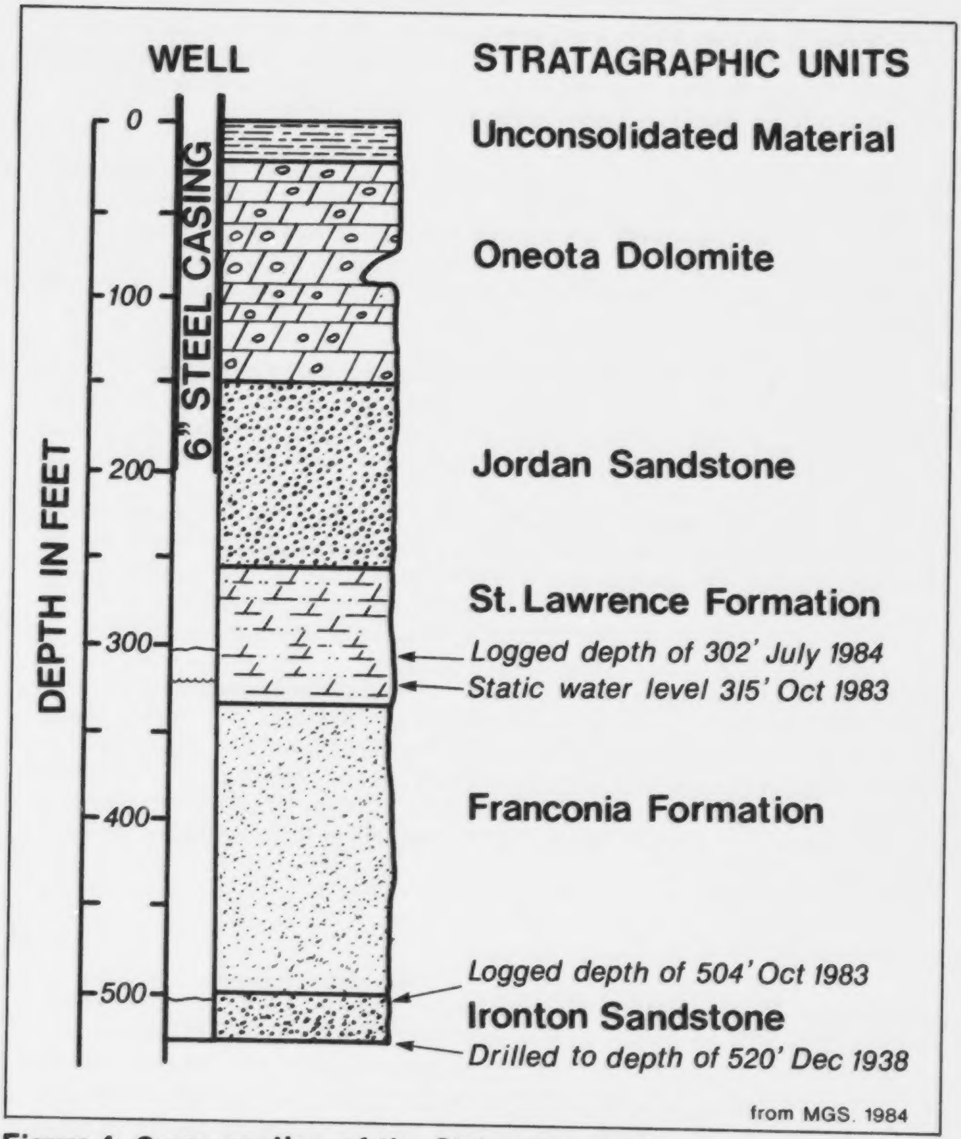
[1] Based on static water level, casing length, and bottom unit.

[2] P = Prairie du Chien, J = Jordan, F = Franconia, I = Ironton, G = Galesville, M = Mt. Simon

D = Dry

? = Unknown

from Bergstrom and Olsen, 1984


WELL
STRATAGRAPHIC UNITS
DEPTH IN FEET
0
100
200
300
400
500
6" STEEL CASING
Unconsolidated Material
Oneota Dolomite
Jordan Sandstone
St. Lawrence Formation
Logged depth of 302' July 1984
Static water level 315' Oct 1983
Franconia Formation
Logged depth of 504' Oct 1983
Ironton Sandstone
Drilled to depth of 520' Dec 1938

from MGS. 1984

Figure 4. Cross section of the Siebenaler Well

The Siebenaler Well is especially noteworthy. It was originally drilled in December of 1938, to a depth of 520 feet, with 200 feet of 6 inch casing set through the Oneota into the Jordan. The borehole was left uncased from the Jordan to the Ironton (see Figure 4). The depth and drilling date were recorded on the cement curbing surrounding the well at the surface.

The MGS first logged it (using gamma) on October 27, 1983, at which time the depth was recorded as being 504 feet with the static water level at a depth of 315 feet. On July 12, 1984 the MGS ran a second log, a caliper log, to learn more about the well's construction and jointed bedrock units. At that time the well was found to be 302 feet deep with a static water level of 301 feet (Figure 4). This suggests that the borehole had collapsed at a level within the St. Lawrence Formation, a dolomitic siltstone with some shale partings (Mossler and Book, 1984).

It is not known whether the collapse occurred coincidentally between the first and second log runs or if the first run affected the integrity of the borehole. Hult (1979), commenting on multiaquifer wells, wrote that, "...where an incompetent rock unit such as a shale aquitard separates the aquifers, the well bore may collapse in response to the flow and the weight of the overlying rocks."

A second explanation for the collapse is related to the bottom portion of the Jordan Sandstone. The lower third is very fine-grained and locally poorly cemented (Thiel, 1944). Since this portion of the Siebenaler Well is uncased, it is possible that the Jordan collapsed into the borehole.

Locally, as at the Siebenaler Well, the Prairie du Chien-Jordan aquifer is dry. However, flows of surface water, prompted by heavy late autumn rains and the spring run-off, may have caused the static water level to fluctuate within the St. Lawrence. This in turn could have eroded the incompetent layers and collapsed that portion of the borehole.

The Siebenaler Well is important in that it is a documented case of a collapsed, multiaquifer well. In many instances, conversations with local residents and former land owners revealed that many of the now collapsed and plugged abandoned wells on DNR property were open within the last few years. "Multiaquifer wells are unstable" (Hult, 1979). Collapsed, multiaquifer wells will present difficulties in achieving a water-tight seal when abandoning old wells.

Conclusions

Previous studies by the MDH (1976) and Hult (1979) showed that abandoned wells and multiaquifer wells are a problem of great magnitude in southeastern Minnesota. A total of 113 abandoned wells were located on DNR land; geophysical findings in 15 of them indicate that 67% of those wells had collapsed since being drilled and 47% were of multiaquifer construction.

Since 70% of the located wells had thinwall casing, which usually only extends through unconsolidated material, they are improperly constructed, unstable and possibly utilized more than one aquifer. Over 11,000 abandoned wells exist in the southeast corner of Minnesota, so to apply the percentages established from Table 2 would indicate that an estimated 5,200 abandoned water wells are multiaquifer, 7,400 are unstable and may collapse if they have not already done so.

Ground water quality in southeastern Minnesota is of vital concern to residents of this agricultural region. For the protection of this resource, the abandoned water wells that are multiaquifer in construction should be properly sealed. Those multiaquifer wells which have already collapsed may have to be redrilled for placement of a water-tight seal.

The problem of abandoned wells and their potential for speeding the spread of contaminants will not go away if ignored. In fact, it is growing: as the population of the region grows, more wells are being drilled, more septic systems are being placed, and more agricultural chemicals are being used.

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Biographical Sketch

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Session VI
Dealing with Hazardous and Non-Hazardous Wastes in Landfills and Impoundments

REGULATORY STRATEGY GOVERNING THE DISCHARGE OF HAZARDOUS AND NONHAZARDOUS WASTE TO LAND IN CALIFORNIA

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Abstract

Revised regulations governing the discharge of waste to land for treatment, storage, or disposal were adopted by the California State Water Resources Control Board on May 1, 1984. The regulations cover discharges of hazardous waste, "designated" wastes (e.g., nonhazardous brines), and nonhazardous solid waste to landfills, waste piles, surface impoundments, and land treatment units. Regulations governing discharges of hazardous waste were developed as part of California's application to conduct a state hazardous waste control program equivalent to the federal program authorized by the Resource Conservation and Recovery Act.

The revised regulations establish classification criteria based on the geologic setting of each waste management unit. Provisions governing closure require dischargers to maintain containment and monitoring systems as long as wastes could affect water quality. Site classification criteria for hazardous waste facilities require units to be located where they are isolated from waters of the state by natural features with low permeability. Combined with construction standards which are more stringent than federal standards, the siting criteria result in redundant containment capability for hazardous wastes.

Class I units (hazardous waste) are required to have natural geologic isolation, a clay liner, a synthetic liner, and a leachate collection and removal system. Class II units (designated waste) are required to have either natural geologic isolation or a clay liner. Class I and most Class II surface impoundments must be double lined. Class III landfills (nonhazardous solid waste) must have either appropriate natural characteristics to ensure no degradation of water quality or a clay liner.

Increased water quality monitoring in both the saturated and unsaturated zones is required in the new regulations. California's ground

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water monitoring program is more stringent than the ground water protection program required by the federal Hazardous Waste Management System (HWMS) regulations. HWMS concepts, such as "compliance monitoring", "alternate concentration limit", and "point of compliance" have been discarded or modified to conform to the State Board's "zero discharge" performance standard. Unsaturated zone monitoring is required for all types of units where feasible given the distance to ground water, the permeability of natural geologic materials, and the ease of installation of monitoring devices.

Introduction

The State Water Resources Control Board (State Board), the California agency charged with protection of ground and surface water, recently adopted new regulations governing the discharge of waste to land. The new regulations control treatment, storage, and disposal of hazardous and nonhazardous waste in landfills, waste piles, surface impoundments, and land treatment units. The regulations also include special provisions for waste generated in confined animal facilities (not discussed in this paper) and for mining waste (discussed elsewhere in these proceedings).

California's previous land disposal regulations, also known as Subchapter 15, were revised for several reasons. The old regulations were implemented by regional water quality control boards according to guidelines prepared by the State Board's technical staff to interpret the regulations. This process resulted in confusion, inconsistencies, and uncertainty in the regulated community. Relevant provisions of the federal Hazardous Waste Management System (HWMS) regulations needed to be incorporated into the State Board's regulations as part of California's application to receive federal authorization to conduct a Resource Conservation and Recovery Act (RCRA)-equivalent program. State Board staff also wished to incorporate into the revised regulations requirements based on the latest technological information on waste containment as well as lessons learned from past experience at waste sites in California.

The new regulations continue to implement State Board Resolution No. 68-16, "Statement of Policy with Respect to Maintaining High Quality of Waters in California". This resolution, known informally as the "nondegradation policy", requires that the "disposal of wastes into waters of the State shall be so regulated as to achieve highest water quality consistent with maximum benefit to the people of the State". Existing quality of water which is higher than that needed for identified beneficial uses must be maintained unless it can be shown that the reduction in water quality will benefit all the people of the State. To meet the criteria in this resolution, it was deemed necessary to require full containment of hazardous and designated wastes for as long as such wastes pose a threat to water quality.

The regulations described here were adopted by the State Board on May 1, 1984 after extensive public comment. The regulations are undergoing review for compliance with procedural requirements by other state agencies at the time of this writing.

Waste and Waste Management Unit Classification

The new regulations divide waste which might be discharged to land into three categories: hazardous, designated, and nonhazardous solid waste. Hazardous waste is that waste which must be managed as hazardous according to standards promulgated by the California Department of Health Services (DHS). DHS is in the process of developing a procedure, the California Assessment Manual (CAM), which uses techniques such as bioassays and leaching tests to determine the degree of hazard posed by a particular waste.

Designated wastes are nonhazardous wastes which need to be fully contained, usually because of their high salt or water content. This category also includes "manageable" hazardous wastes, those low-risk hazardous wastes which qualify for a variance from hazardous waste management requirements pursuant to DHS regulations. Such wastes primarily contain heavy metals.

TABLE 1
SUMMARY OF WASTE MANAGEMENT STRATEGIES FOR DISCHARGES OF WASTE TO LAND

Waste Category	Waste Management Strategy	Waste Management Unit	
		Class	Type
Liquid Hazardous	Full Containment	I	Surface Impoundment
Solid Hazardous	Full Containment	I	Landfill
Dry Solid Hazardous	Full Contaimment	I	Waste Pile
Liquid Designated (including undewatered sludge)	Full Containment	II	Surface Impoundment
Solid Designated	Full Containment	II	Landfill
Dry Solid Designated	Full Containment	II	Waste Pile
Nonhazardous Solid Waste (including dewatered sludge and acceptable incinerator ash)	Protect Beneficial Uses	III	Landfill

Nonhazardous solid waste is a residual category which includes domestic and industrial wastes that can be managed without complete containment. Nonhazardous ashes and dewatered sludge from sewage treatment may be mixed with nonhazardous solid waste.

Waste management units have been divided into Class I, Class II, and Class III, as shown on Table 1. This table displays the requirements applicable to new facilities; existing facilities may have reduced requirements. Wastes in any category may be discharged at waste manage ment units with higher levels of containment ability. Wastes suitable for land treatment in any category may be discharged at land treatment facilities.

Control of liquids is an important part of these regulations. Discharge of liquid waste is generally not permitted at hazardous waste landfills, although such discharge may be permitted until December 31, 1989 if no alternative disposal is available, if the landfill is equipped with a liner and leachate collection and removal system, and if a 5:1 solids-to-liquids ratio is maintained. At landfills receiving designated waste (Class II), liquid may not be discharged in excess of the moisture-holding capacity as a result of waste managment operations. Similarly, discharge of liquids at Class III landfills is not permitted unless the discharger can demonstrate that such discharge will not exceed the present or future moisture-holding capacity of the landfill.

Dewatered nonhazardous sewage or water treatment sludge may be discharged at a Class III landfill if: (1) the landfill is equipped with a leachate collection and removal system; (2) the sludge contains at least 20 percent solids if primary sludge or at least 15 percent solids if secondary, mixed, or water treatment sludge; and (3) a minimum solids-to-liquid ratio of 5:1 by weight is maintained. The percent solids requirement is based on the usual moisture contents achieved using conventional treatment processes. The minimum solids-to-liquids ratio is based on the results of Southern California experiments on sludge-refuse mixtures and resultant leachate generation during waste compaction. Because the moisture retention capacity of landfilled waste varies greatly with the degree of compaction and the contents of the refuse, a leachate collection and removal system was deemed necessary to ensure that excess fluid does not escape the unlined landfill in amounts that overwhelm the attenuation capability of the soil.

Siting Criteria

Unlike the federal HWMS, the new State Board regulations contain siting criteria which emphasize natural site characteristics. Siting parameters include geologic setting, rapid geologic change, and tidal waves, as well as flooding and ground rupture. The criteria for new sites are summarized on Table 2; existing facilities, in some cases, have reduced requirements.

TABLE 2

SITING CRITERIA FOR NEW WASTE MANAGEMENT UNITS

Site Characteristics	Waste Management Unit Classification		
	New Class I	New Class II	New Class III
Geologic Setting	Maximum attainable isolation from ground water: substantial thickness, permeability $\leq 1\times10^{-7}$ cm/sec.	Substantial isolation from ground water: substantial thickness, permeability $\leq 1\times10^{-6}$ cm/sec (or liner system).	Adequate separation from ground water: characteristics other than permeability will be considered.
Flooding	Outside of 100-year floodplain.	Must protect unit from the hazard.	
Ground Rupture	200' setback from known Holocene fault.	200' setback from known Holocene fault.	Not located on known Holocene fault.
Rapid Geologic Change	Outside subject area (potential to impair containment).	Must protect unit from the hazard.	
Tidal Waves	Outside subject coastal areas.	Must protect unit from the hazard.	No siting restriction.

Geologic Setting

The geologic setting criterion requires new and existing Class I treatment, storage, and disposal units to be immediately underlain by natural geologic materials having a permeability no greater than 1×10^{-7} cm/sec. Natural geologic materials having a permeability no greater than 1×10^{-6} cm/sec and immediately underlying a Class II unit may be substituted for a required liner. In either case, such natural materials must be thick enough to preclude vertical movement of fluid, including waste or leachate. Lateral movement of fluid can be prevented by either natural or artificial means. These requirements make the natural material the final backup containment system. Artificial prevention of lateral movement is allowed since such containment structures are easier to install and maintain than any such containment structures beneath the waste.

Class III landfills are not required to satisfy stringent containment requirements, but must be sited where soil characteristics, the distance from waste to ground water, and other factors will ensure adequate attenuation of pollutants derived from the waste before such pollutants can have an adverse effect on the quality of receiving surface and ground waters.

Requirements for impermeable containment features also do not apply to land treatment units. Such units must have site characteristics suitable for land treatment as shown by a demonstration of complete transformation, degradation, and immobilization of wastes within a treatment zone extending 5 feet below the land surface.

All types and classs of units are required to have a 5-foot separation between the base of waste and the top of the capillary fringe of underlying ground water. This separation was chosen to ensure that direct contact between waste and moisture continuous with the water table is prevented and to allow installation of unsaturated zone monitoring devices beneath the units for leak detection. Except in the case of land treatment, separation may be naturally or artificially maintained (e.g., by pumping). Because land treatment units have no containment features and proper management alone prevents waste from penetrating beyond the treatment zone, the risk to ground water was felt to be too great to allow dischargers to maintain the required separation between wastes and ground water artificially.

Flooding and Tidal Waves

The 100-year floodplain has been used as the siting criterion for flooding. Siting outside these floodplains is required for new and existing Class I disposal units pursuant to existing regulations. Other types and classes of units must be protected from the 100-year flood. Siting outside the 100-year floodplain is and has been required because of the long lifespan of hazardous waste. It may be impossible to provide flood protection for the full time that a buried waste poses a threat to water quality.

Tidal waves are tsunamis, seiches, and surges. New Class I disposal facilities must be located outside areas subject to tidal waves for reasons analagous to those for floodplain siting. Other types of Class I facilities must be protected against such occurrences. This criterion does not apply to other classes of units.

Ground Rupture and Rapid Geologic Change

To protect containment features from ground rupture, a 200-foot setback from any known Holocene fault is required for new Class I and II units, existing Class I disposal units, and reconstructions or expansion of existing Class I or II units. New Class III units or expansions of existing units cannot be located on a known Holocene fault.

The rapid geologic change criterion uses the presence of landslides, subsidence, settlement, or other changes which occur rapidly relative to the combined active life and post-closure maintenance period of the waste management unit. Such changes are of concern when they threaten the integrity of containment, drainage, and monitoring facilities. Siting outside of areas subject to such change is required for new and existing Class I disposal units pursuant to both the new and existing regulations. Other types and classes of units must protect against such change.

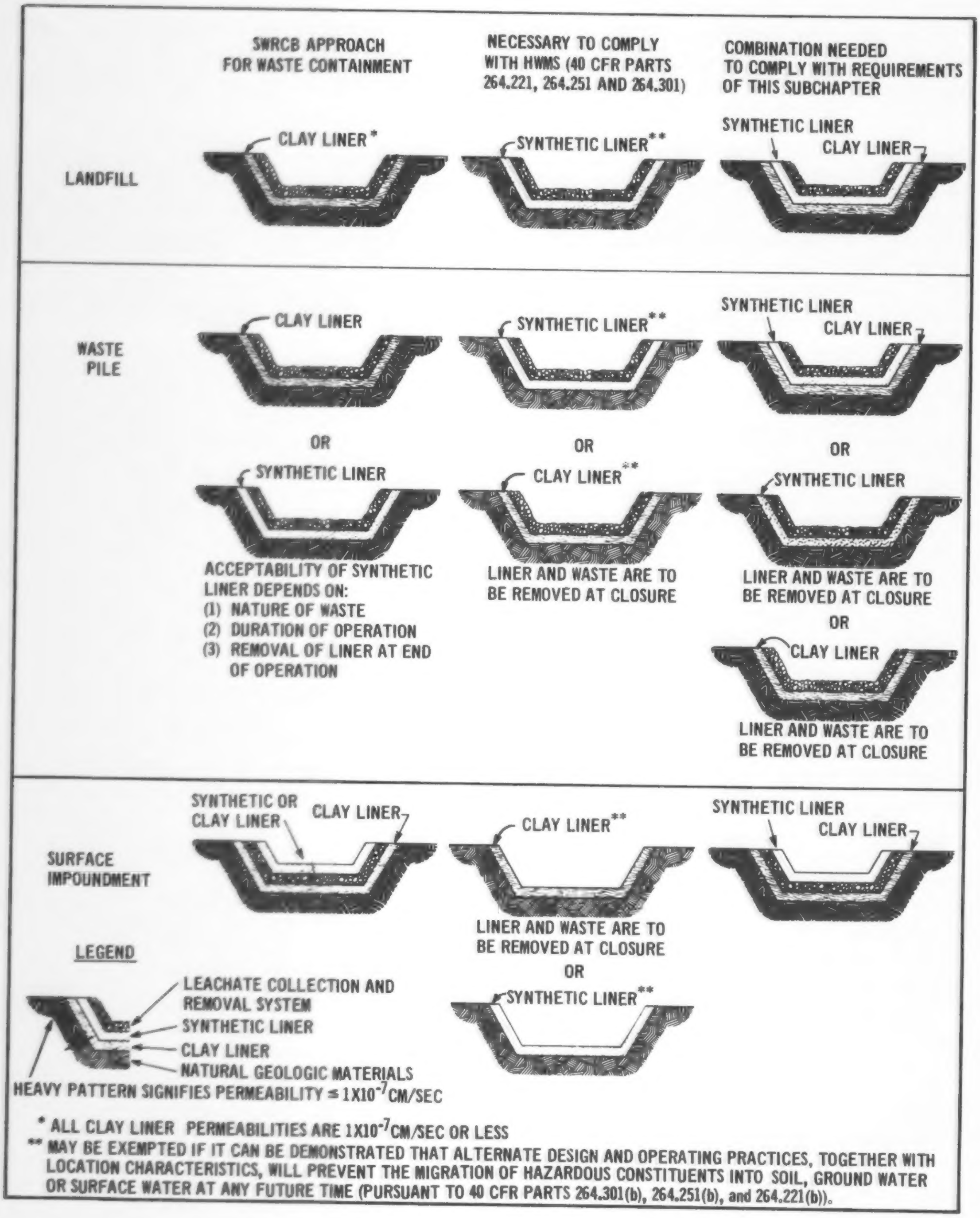
Construction Standards

Specific construction standards are contained in the new regulations. The old regulations were performance-oriented and have had a poor record in preventing contamination. Assessments of site suitability under the guidelines which had been developed to assist regional boards were necessarily subjective. It is difficult to adequately test a new design to determine whether or not such a design ensures prospective compliance with a performance standard.

There may be occasions when it is not feasible to comply with the prescriptive standards in the new regulations because the prescribed standard is unreasonably burdensome, too expensive, or ineffective. In these cases, regional boards may approve a specific engineered alternative which is consistent with the performance goal addressed by the particular construction standard and which affords equivalent protection against water quality degradation.

Construction details for new landfills, waste piles, and surface impoundments are shown in Figures 1 and 2, and summarized in Table 3. For Class II units, natural geologic materials immediately underlying the unit and having a permeability no greater than $1x10^{-6}$ cm/sec may be substituted for the required clay liner for landfills and waste piles, and for the required outer clay liner for double-lined surface impoundments. The amount of retrofitting required for an existing facility will depend on the results of expanded ground water and unsaturated zone monitoring required in the new regulations.

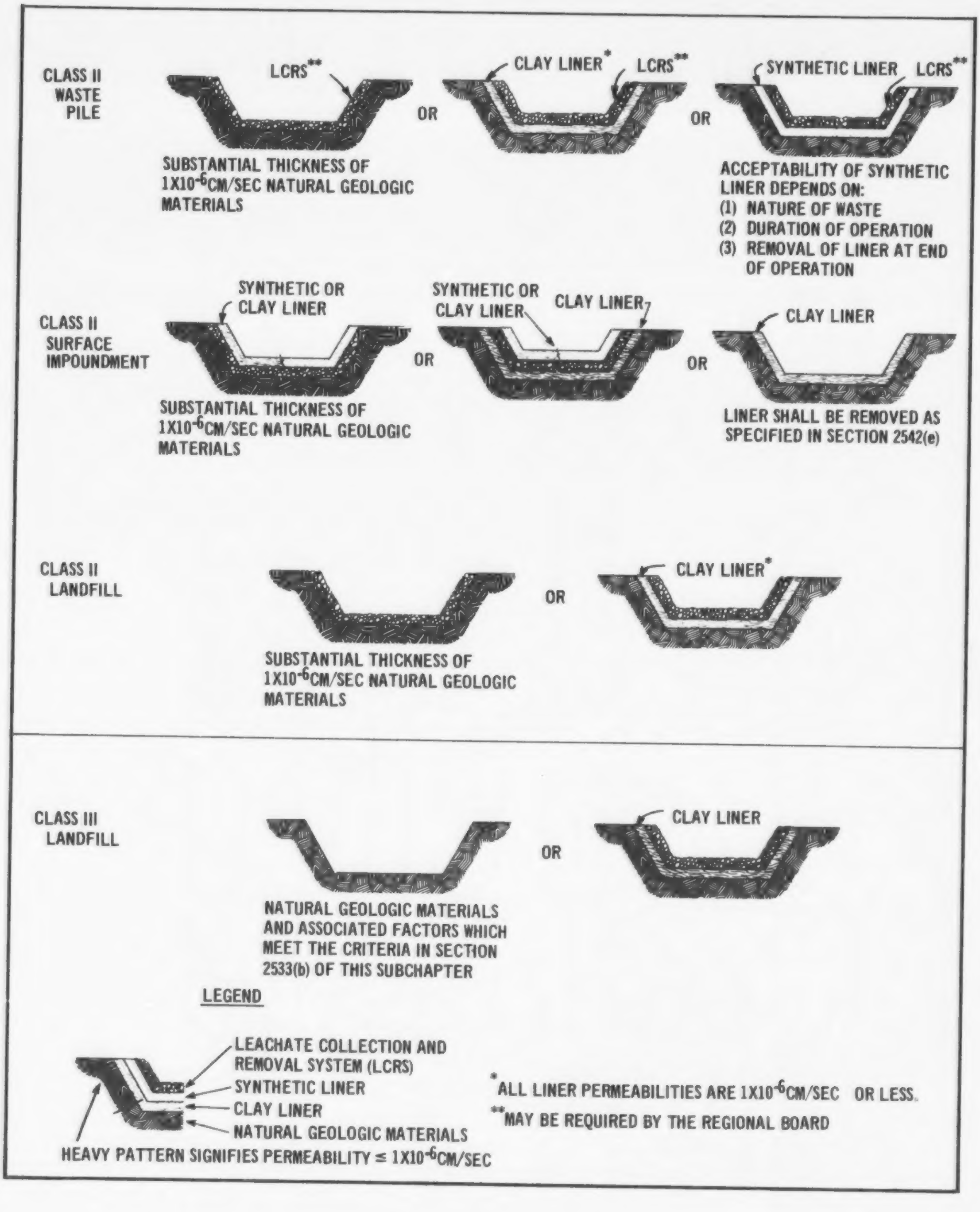
Primary reliance for containment has been placed on clay liners in the new regulations (the term "clay liner" refers to any earthen liner which meets the stated permeability requirements). Historically, synthetic liners have not performed well in California. No long-term data are available to gauge the durability of synthetic liners. Inadequate seaming of synthetic liners was seen by State Board staff as one of the greatest drawbacks to effective waste containment by synthetic liners. Clay liners by contrast have a more extensive performance record and are installed using commonly understood construction practices.


SWRCB APPROACH FOR WASTE CONTAINMENT
NECESSARY TO COMPLY WITH HWMS (40 CFR PARTS 264.221, 264.251 AND 264.301)
COMBINATION NEEDED TO COMPLY WITH REQUIREMENTS OF THIS SUBCHAPTER
LANDFILL
CLAY LINER*
SYNTHETIC LINER**
SYNTHETIC LINER
CLAY LINER
WASTE PILE
CLAY LINER
SYNTHETIC LINER**
SYNTHETIC LINER
CLAY LINER
OR
OR
OR
SYNTHETIC LINER
CLAY LINER**
SYNTHETIC LINER
ACCEPTABILITY OF SYNTHETIC LINER DEPENDS ON:
(1) NATURE OF WASTE
(2) DURATION OF OPERATION
(3) REMOVAL OF LINER AT END OF OPERATION
LINER AND WASTE ARE TO BE REMOVED AT CLOSURE
LINER AND WASTE ARE TO BE REMOVED AT CLOSURE
OR
CLAY LINER
LINER AND WASTE ARE TO BE REMOVED AT CLOSURE
SURFACE IMPOUNDMENT
SYNTHETIC OR CLAY LINER
CLAY LINER
CLAY LINER**
SYNTHETIC LINER
CLAY LINER
LINER AND WASTE ARE TO BE REMOVED AT CLOSURE
OR
SYNTHETIC LINER**
LEGEND
LEACHATE COLLECTION AND REMOVAL SYSTEM
SYNTHETIC LINER
CLAY LINER
NATURAL GEOLOGIC MATERIALS
HEAVY PATTERN SIGNIFIES PERMEABILITY ≤ 1X10⁻⁷ CM/SEC

* ALL CLAY LINER PERMEABILITIES ARE 1×10^{-7} CM/SEC OR LESS

** MAY BE EXEMPTED IF IT CAN BE DEMONSTRATED THAT ALTERNATE DESIGN AND OPERATING PRACTICES, TOGETHER WITH LOCATION CHARACTERISTICS, WILL PREVENT THE MIGRATION OF HAZARDOUS CONSTITUENTS INTO SOIL, GROUND WATER OR SURFACE WATER AT ANY FUTURE TIME (PURSUANT TO 40 CFR PARTS 264.301(b), 264.251(b), and 264.221(b)).

FIGURE 1. Summary of clay and synthetic liner requirements for Class I landfills, waste piles, and surface impoundments


CLASS II
WASTE
PILE
LCRS**
OR
CLAY LINER*
LCRS**
OR
SYNTHETIC LINER
LCRS**
SUBSTANTIAL THICKNESS OF
1X10-6CM/SEC NATURAL GEOLOGIC
MATERIALS
ACCEPTABILITY OF SYNTHETIC
LINER DEPENDS ON:
(1) NATURE OF WASTE
(2) DURATION OF OPERATION
(3) REMOVAL OF LINER AT END
OF OPERATION
CLASS II
SURFACE
IMPOUNDMENT
SYNTHETIC OR
CLAY LINER
OR
SYNTHETIC OR
CLAY LINER
CLAY LINER
OR
CLAY LINER
SUBSTANTIAL THICKNESS OF
1X10-6CM/SEC NATURAL GEOLOGIC
MATERIALS
LINER SHALL BE REMOVED AS
SPECIFIED IN SECTION 2542(e)
CLASS II
LANDFILL
OR
CLAY LINER*
SUBSTANTIAL THICKNESS OF
1X10-6CM/SEC NATURAL GEOLOGIC
MATERIALS
CLASS III
LANDFILL
OR
CLAY LINER
NATURAL GEOLOGIC MATERIALS
AND ASSOCIATED FACTORS WHICH
MEET THE CRITERIA IN SECTION
2533(b) OF THIS SUBCHAPTER
LEGEND
LEACHATE COLLECTION AND
REMOVAL SYSTEM (LCRS)
SYNTHETIC LINER
CLAY LINER
NATURAL GEOLOGIC MATERIALS
HEAVY PATTERN SIGNIFIES PERMEABILITY ≤ 1X10-6CM/SEC

*ALL LINER PERMEABILITIES ARE $1X10^{-6}$CM/SEC OR LESS.

**MAY BE REQUIRED BY THE REGIONAL BOARD

FIGURE 2. Summary of liner requirements for Class II waste piles and surface impoundments and Class II and III landfills

TABLE 3
SUMMARY OF CONSTRUCTION STANDARDS FOR NEW WASTE MANAGEMENT UNITS

Waste Mgmt. Unit Classification	Type of Waste Management Unit	Clay Liner	Synthetic Liner	Leachate Collection and Removal System	Interim Cover	Subsurface Barriers as needed		Capacity of Precipitation and Drainage Control Facilities	Seismic Design
						Cutoff Walls	Grout Curtains		
Class I	Landfill	required, $\leq 1x10^{-7}$ cm/sec	required	required, blanket type	required	$\leq 1x10^{-7}$ cm/sec	$\leq 1x10^{-7}$ cm/sec	probable maximum precipitation	withstand maximum credible earthquake
	Surface Impoundment	required, outer liner, $\leq 1x10^{-7}$ cm/sec	required, inner liner	required, blanket type		$\leq 1x10^{-7}$ cm/sec	$\leq 1x10^{-7}$ cm/sec		
	Waste Pile	Either liner alone may be allowed depending on waste and duration of use. Both liners required to close unit in place.		required, blanket type	required	$\leq 1x10^{-7}$ cm/sec	$\leq 1x10^{-7}$ cm/sec		
Class II	Landfill	required $\leq 1x10^{-6}$ cm/sec	not required	required, blanket type	required	$\leq 1x10^{-6}$ cm/sec	$\leq 1x10^{-6}$ cm/sec	1000-year, 24-hour precipitation	
	Surface Impoundment	Either a single replaceable clay liner or a double-lined system (inner liner synthetic or clay, outer liner clay) required.		required with double liner, blanket type		$\leq 1x10^{-6}$ cm/sec	$\leq 1x10^{-6}$ cm/sec		
	Waste Pile	Synthetic liner may be allowed depending on waste and duration of use. Clay liner required to close unit in place.		may be required, blanket type	may be required	$\leq 1x10^{-6}$ cm/sec	$\leq 1x10^{-6}$ cm/sec		
Class III	Landfill	required if natural conditions unsuitable, $1x10^{-6}$ cm/sec	not required	required if liner is required, blanket or dendritic	required	$\leq 1x10^{-6}$ cm/sec	$\leq 1x10^{-6}$ cm/sec	100-year, 24-hour precipitation	withstand maximum probable earthquake

In contrast to the federal HWMS, double liners (inner liner synthetic, outer liner clay) are required for new hazardous waste (Class I) surface impoundments. A leachate collection and removal system is required between the liners to detect leaks through the inner liner and to prevent the buildup of head on the outer liner until a leaking inner liner can be replaced. The design requirements for impoundments reflect the increased head which could force fluids through the federally-mandated single synthetic liner at points of weakness. Reliance on a single liner, especially a synthetic liner, was one of the deficiencies in the federal HWMS which the State Board was determined to correct. The overall Class I surface impoundment design provides the redundant protection to ground water by requiring two liners separated by a leachate collection and removal system, and backed up by natural geologic material of low permeability required under the siting standards. Similar redundancy has been incorporated into the requirements for other Class I and for Class II units to ensure complete waste containment.

Monitoring

The monitoring requirements in the new regulations are based on the ground water protection provisions of the federal HWMS. The new regulations contain no equivalent to interim status monitoring pursuant to 40 CFR Part 265, but instead require monitoring correlative to Part 264. Although interim status parameters such as total organic carbon (TOC) and total organic halogen (TOX) are inexpensive to analyze for, the results of such analyses are difficult to interpret and are often inconclusive.

The new regulations include mandatory ground water monitoring at all units, mandatory unsaturated zone monitoring at Class I land treatment units, and unsaturated zone monitoring at other Class I units and at Class II units as feasible. The feasibility of unsaturated zone monitoring at a specific site is determined after considering the distance to ground water, permeability of natural materials, and the ease of installation of unsaturated zone monitoring devices.

In contrast to the federal HWMS, leakage of waste from a facility until "alternative concentration limits" are reached in ground water is not allowed. Background water quality is the standard against which other concentrations are measured. For this reason, the stage in the monitoring in the federal HWMS after a leak is detected, "compliance monitoring" was renamed "verification monitoring" in the new regulations. Verification monitoring encompasses those actions needed to confirm or deny a suspected leak, to determine the extent of contamination, and to develop a complete corrective action program. The other monitoring programs in the new regulations, "detection monitoring" and "corrective action", are correlative to the federal HWMS and encompass routine monitoring at nonleaking units and cleanup activities, respectively.

The federal concept of "point of compliance" has been modified to include those points at which unsaturated zone monitoring takes place. The new regulations do not confine monitoring to the "uppermost aquifer" but require earliest detection of contamination. Similarly, the responsibility of the discharger to cleanup contamination does not end at the fence line but includes offsite investigation and corrective action as necessary.

Proposals for monitoring networks which meet the criteria in the new regulations must be developed for existing sites within 6 months after the new regulations take effect. Waste discharge requirements for all existing units must be reviewed and units must be reclassified as needed within 5 years after the regulations take effect. At the time of the review, the results of the monitoring program will be reviewed to determine the need for additional containment. The need for retrofitting of existing facilities with containment structures required for new facilities will depend largely on the results of ground and unsaturated zone monitoring.

Closure and the Post-Closure Maintenance Period

The most important difference between the federal HWMS and the new regulations is the length of the post-closure maintenance period. Instead of the 30-year period specified by the federal regulations, the post-closure maintenance period for all classes of waste management units in California must extend as long as wastes pose a threat to water quality. This long protection period was seen as critical given the long lifespans of certain wastes.

The cover design for all classes of landfills and for waste piles and surface impoundments closed in place requires 2 feet of foundation materials which may be contaminated soil or similar waste material with suitable structural properties. One foot of low-permeability earth materials having a permeability of 1×10^{-6} cm/sec or less, or equal to the permeability of any bottom liner or natural geologic material underlying the unit must be installed above the foundation layer. The low-permeability layer must be covered by a rooting layer for vegetation (minimum thickness of 1 foot). If the unit is a Class I facility, a synthetic cover is required in addition to the earthen low-permeability layer to comply with the federal regulations.

Conclusions

The new regulations specify construction standards and siting criteria for landfills, surface impoundments, waste piles, and land treatment units receiving hazardous, designated, and nonhazardous solid waste. In the case of hazardous and designated waste, a combination of liners, leachate collection and removal systems, and low-permeability natural geologic materials are intended to provide complete containment for discharged wastes as long as such wastes pose a threat to water quality.

Biography of Presenting Author

Ms. Herbst received a B.S. degree in geology from the University of California, Davis, in 1978. Since joining the Hydrogeology Section of the State Water Resources Control Board as an Engineering Geologist, she has advised the regional water quality control boards on the cleanup of leaking underground tanks, on landfill expansions, on waste injection wells, and on remedial action at the abandoned Leviathan Sulfur Mine. Before joining the State Board, she worked for the California Department of Water Resources, authoring several bulletins on the ground water resources of Sonoma County.

CONTRARY WASTE SITE CHARACTERISTICS --
GOOD IS BAD, BAD IS GOOD

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ABSTRACT

The evaluation of hazardous waste sites has been hampered because the hydrogeologic factors involved are complexly interrelated and often contrary. These factors are so intertwined that no site can have favorable features only, without any unfavorable ones. For example, low permeability for the purpose of waste containment is normally favorable but is likely to be associated with a high water table, which is unfavorable. Seven key hydrogeologic factors -- Permeability, Sorption, Water Table Depth, Water Table Gradient, Stream Density, Topography, and Ground Distance -- may be strategically used in assessments because they are important in determining the movement of contaminants, they can be readily applied, and they permit useful inferences about other factors. Though these factors show many direct and inverse relationships among each other, they are inescapably a part of any evaluation system. A careful weighting of their tendencies and counter-tendencies leads to optimal assessment of waste sites.

INTRODUCTION

Optimal hydrogeologic characteristics for hazardous waste sites have been recognized for more than 25 years, the early studies being prompted by the development of radioactive wastes. Then, why can't we move forward in choosing the best waste sites? The truth is that the waste site characteristics, or factors, are complexly interrelated and entwined in a somewhat similar way as are the factors directed toward a healthy national economy. The purpose of this paper is to identify some of the hydrogeologic factors, to demonstrate their interrelations, and to suggest ways of handling the complexities.

The contrary waste site characteristics, caused by the natural interdependence of hydrogeologic factors, are seldom fully explored. Efforts by regulatory agencies to fully use hydrogeologic factors in regulations dealing with hazardous wastes have been thwarted by objections arising from conflicting tendencies and counter-tendencies. Regulations now emphasize performance standards and mostly shy away from the full intricacies of hydrogeology. Renewed studies of these factors will surely lead to a better integration of the hydrogeologic framework into appropriate regulations.

KEY HYDROGEOLOGIC FACTORS

The factors to be considered are: Permeability, Sorption, Water Table Depth, Water Table Gradient, Stream Density, Topography, and Ground Distance (between a source of contamination and a well or stream). These factors were selected because (1) they are important to the assessment, (2) they can be readily applied, and (3) they permit useful inferences that reduce the need for intensive data in many cases. It is noted that such factors as climate, evapotranspiration, and background water quality are not listed. Whereas these factors, as well as others, are important if a study is made in a certain context, they do not have the full interrelated usefulness that the factors listed above have. Fortunately, many unlisted factors are already embodied to a certain extent in the factors that are used here. For example, the type of climate can be inferred from approximate values of such factors as Water Table Depth and Stream Density.

A seeming contradiction may appear concerning the hydrogeologic factors. As indicated in the Introduction, the complex interrelated factors result in a somewhat muddled and difficult struggle with objective evaluation of the hydrogeologic characteristics of waste sites. Yet, the interrelations of the factors may be useful. Throughout this discussion it will be noted that if something is known about two hydrogeologic characteristics, information about a third characteristic may be inferred. This paper does not fully explore the inferences that can be made of the interrelations.

Permeability

(A) General Comments -- Gravels and clean coarse sands, as well as some fractured and some soluble rocks, are highly permeable. Clays and some dense, poorly fractured rocks have low permeability. The most favorable situation for a waste site may be where thick clay beds underlie the site. Rock permeability in the form of fractures and solution openings is likely to be unacceptable for a site.

(B) Advantageous and Disadvantageous Aspects -- High permeability near the surface allows for infiltration of contaminated water. This is favorable on the one hand because surface contamination and runoff is prevented or reduced, but unfavorable on the other hand because the contaminants may move quickly to the ground water, before dilution or decay reaches an acceptable stage. High permeability is generally associated with a deep water table, leading to better opportunities for contaminants to attenuate in the unsaturated zone. Low permeability near the surface is favorable because the movement of contaminants is retarded, allowing additional time for attenuation in the ground. However, low surface permeability causes rejection of contaminated water and helps create problems of contaminants on the ground surface and in streams. Low permeability may result in a shallow water table and thus less opportunity for attenuation in the unsaturated zone.

Sorption

(A) General Comments -- Sorption is a desirable action if contaminants move through clay and other sorptive materials. However, because of the inverse relation of sorption with permeability, contaminants tend to move around clay beds if permeable pathways are available. The most favorable situation may be a little clay mixed with sand.

(B) Advantageous and Disadvantageous Aspects -- On the positive side, clay, to varying degrees, has the capacity to sorb many contaminants and thus to prevent or retard them from moving at the same rate as the associated ground water. Poor sorptive materials are commonly very permeable. For the most part, sorption is inversely related to permeability, and other aspects that could be listed here have already been covered above in "Permeability".

Water Table Depth

(A) General Comments -- For this discussion, a "shallow" water table is less than approximately 15 feet below land surface, and a "deep" water table is more than 75 feet below land surface. Shallow water tables are common in humid regions and deep water tables in arid regions. A deep water table is desirable for a waste site. A water table that rises during its high stage to the main body of wastes or near the land surface may be unacceptable.

(B) Advantageous and Disadvantageous Aspects -- A deep water table has several advantages. First, contaminants can be infiltrated into the extensive unsaturated zone, reducing overland flow or the emergence of contaminants at the land surface. Second, extensive attenuation of contaminants can occur in the unsaturated zone, relative to that where the water table is shallow. Third, some contaminants may never reach a deep water table and thus remain in the unsaturated zone, where their potential for harm may be less than in the saturated zone. A deep water table can be disadvantageous because monitoring and control of contaminated zones would be difficult as a result of the great depths beneath the surface. Conversely, a shallow water table has mostly disadvantageous features, especially in allowing contaminants to either emerge at the land surface or to rather quickly reach the saturated zone, where lateral dispersion is predominant.

Water Table Gradient

(A) General Comments -- Gradients are easy to determine where water-table maps are available. However, good inferences lead to acceptable approximations of water-table gradients in most cases. Steep land surfaces in combination with low permeability and humid climates tend to yield steep water-table gradients.

(B) Advantageous and Disadvantageous Aspects -- A water-table gradient away from a point of water use is desirable because contamination moves away from that point. If the gradient is toward a water source, a gentle gradient (less than 2 percent) is more acceptable than a steep gradient (more than 5 percent) because of the slower water movement and greater time for contaminant attenuation, unless the gentle gradient is due to high transmissivity. A steep gradient toward a point of water use is almost always undesirable.

Stream Density

(A) General Comments -- For this discussion, a network of "closely spaced" streams has a perennial stream within a mile of all interstream points. If the ground water is contaminated, the stream generally represents a boundary beyond which the contamination may not extend.

(B) Advantageous and Disadvantageous Aspects -- Closely spaced streams limit the areal extent of contaminated zones in the ground. However, in this situation, contaminants may have a short underground course before dispersing into a stream, where they may be objectionable. Sparsely spaced streams may allow for attenuation in the ground and thus greatly reduce or avoid the chance for stream contamination. On the other hand, contaminated zones in the ground may be very large and more nearly permanent where streams are widely spaced, rather than where closely spaced.

Topography

(A) General Comments -- For this purpose, topography of low relief does not have high hills or deep valleys. Topography is an important factor, but its value is more tangibly considered within the interrelations of the factors of Permeability, Water Table Depth, and Water Table Gradient.

(B) Advantageous and Disadvantageous Aspects -- Low relief is commonly associated with a low water-table gradient and, if permeability is low, with slow movement of ground water and thus a slow flow of contaminants. However, low relief is commonly associated with a shallow water table, especially in humid areas; thus there may be a minimum of attenuation in the unsaturated zone. High relief is commonly associated with a deep water table (where permeability is moderate to good). The major disadvantage of a waste site in a region of high relief is that the ground water is generally rapidly moving, which may limit the time for attenuation of contaminants.

Ground Distance (between a source of contamination and a well or stream) 5

(A) General Comments -- Distance (on the ground) from a waste site to a water supply is the most distinct hydrogeologic factor, is easily recorded, and is the factor most often used in site evaluations.

(B) Advantageous and Disadvantageous Aspects -- The farther the site is from a water supply, the less the chance for contamination, all other factors being equal. A short distance is obviously undesirable.

INTERRELATIONS OF THE FACTORS

An evaluation of waste site characteristics, or factors, as shown above, presents useful information. Yet, this listing of key characteristics and trends gives information in a scrambled and unsorted form.

Figure 1 shows some direct and inverse relations between key hydrogeologic factors, as related to waste site assessment. A number of the relations should be regarded as tendencies, rather than clear-cut relationships. These tendencies are, for the most part, indicated by just a single "+" or a single "-" rather than by a double symbol. For example, there is a weak tendency for regions with highly permeable soils to have rather few and widely spaced streams; thus, Permeability and Stream Density are given just a single "-" on the figure. A feature not shown in the figure is whether the relationship is a primary one or a secondary (or indirect) one, whereby a third factor is generally involved. As an example, Topography and Stream Density show a strong direct relationship because, in general, an area of hilly topography contains an abundance of streams in the many valleys, and a flat setting contains few streams. Another primary relationship can be found between Topography and Water Table Gradient -- great topographic relief is usually associated with a high water table gradient. However, Stream Density and Water Table Gradient also show a "++" on the figure. The relationship between these two variables is secondary, or indirect, because it is a third factor (Topography) that is contributing to this positive relationship between Stream Density and Water Table Gradient.

Figure 2 shows whether a "high" or a "low" value of each hydrogeologic factor is a favorable or an unfavorable waste site characteristic. For example, a site underlain by a deep water table (a great Water Table Depth) would be much more favorable than one underlain by a water table almost near the ground surface.

By applying the key information from Figures 1 and 2, especially the latter figure, to a particular site, the overall good and bad site characteristics can be reviewed. It is readily seen that most sites have both good and bad hydrogeologic features and that no site is good in all respects because of the complex interrelations of the factors. The good and bad features rarely cancel out equally. The great range in the overall quality of waste sites can be assessed only be carefully weighting the pertinent factors (LeGrand, 1983).

	GROUND DISTANCE	TOPOGRAPHY	STREAM DENSITY	WATER TABLE GRADIENT	WATER TABLE DEPTH	SORPTION	PERMEABILITY
PERMEABILITY	+	0	−	− −	+ +	− −	
SORPTION	−	0	+	+ +	− −		
WATER TABLE DEPTH	+	+	−	− −			
WATER TABLE GRADIENT	−	+ +	+ +				
STREAM DENSITY	− −	+ +					
TOPOGRAPHY	− −						
GROUND DISTANCE (from waste site to stream)							

+ +	Strong direct relationship
+	Weak direct relationship
0	Little or no relationship
−	Weak inverse relationship
− −	Strong inverse relationship

Figure 1. Interrelationships of key hydrogeologic factors at waste sites. These relationships apply primarily to humid regions.

	HIGH or Great	LOW or Small
PERMEABILITY		☺
SORPTION	☺	
WATER TABLE DEPTH	☺	
WATER TABLE GRADIENT		☺
STREAM DENSITY		☺
TOPOGRAPHY		☺
GROUND DISTANCE (from waste site to stream)	☺	

☺ Favorable situation

☐ Unfavorable situation

Figure 2. Tendencies of key hydrogeologic factors to be favorable or unfavorable at waste sites. The two columns represent high and low values for each factor.

The factors should be weighted in an assessment because certain ones, such as Permeability of the rock materials and Ground Distance from a waste site to the nearest stream, are clearly more important than others. Also, because of the interrelations of the factors, a factor that may seem important in a waste site assessment might have to be discarded because it is closely tied or related (directly or inversely) to another factor. As an example, everyone agrees that the sorption of the rock materials beneath, and in the vicinity of, a waste site is very important in an assessment. Yet, Sorption shows a strong inverse relationship to Permeability (see Figure 1) -- materials with high sorptive capacities tend to be relatively impermeable. Thus, one must pick and choose the factors needed for a waste site assessment very carefully. One would like to have factors that are completely independent of each other for an assessment. Relatively independent factors such as Climate and Human Population living near a waste site have often been used for site assessments. However, they stray from the core of the issue -- the determination of the potential for contamination of ground water by a waste site.

Efforts by the U. S. Environmental Protection Agency (Caldwell et al., 1981) and by some states (Hutchinson and Hoffman, 1984; Michigan Department of Natural Resources, 1984) to weight a group of factors for ranking of sites have been only partly successful. Much of the past effort in weighting of values of factors has been directed to include many factors, some of which are not in every case important; the result has been complex, unwieldy evaluation attempts. Moreover, the failure to focus on the interrelated factors results in a somewhat sterile evaluation that does not easily describe the site. On the other hand, a properly weighted method of site evaluation and ranking takes into consideration the interrelations of hydrogeologic factors and thus presents graphically the good and bad features of a site.

DISCUSSION

Our attempt to find ideal or optimal waste site characteristics is fraught with difficulties. We are faced with complexities much like those of economists dealing with the national economy, who try to weight such sensitive and interrelated factors as the federal deficit, inflation, interest rates, unemployment, and import-export balances. For example, those who argue for cheap imports should reckon with a trade imbalance that leads to unemployment and to an increased federal deficit. Rather than doting on a single aspect of the economy or of waste site characteristics, it is better to spell out, sort, and weight sensibly the key factors.

It has been demonstrated that such site factors as low permeability, high sorptive capacity, deep water table, low hydraulic gradient, and great distance from a waste site to a stream may not be compatible. Thus, it is necessary to make limited concessions with respect to some of the factors and to apply such engineering design features as may be appropriate. Above all, a standardized weighting and rating system applied specifically to the interrelated hydrogeologic factors can demonstrate the best evaluation of a waste site.

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BIOGRAPHICAL SKETCH

Harry E. LeGrand is a consulting hydrogeologist in Raleigh, North Carolina. Prior to 1975, he was a research hydrologist with the U. S. Geological Survey. He has authored more than 80 reports and papers, many of which relate to ground-water contamination. His major interest in the past 25 years has been in hydrogeology related to contaminated zones. He is a past Chairman of the Ground Water Technology Division of the National Water Well Association. His mailing address is: 331 Yadkin Drive, Raleigh, NC 27609.

TRANSPORT OF ORGANIC CONTAMINANTS IN GROUND WATER

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Abstract

A numerical model has been developed to describe the transport of organic contaminants in a two-dimensional, integrated saturated-unsaturated porous medium. The transport problem requires the solution of (1) equations describing simultaneous flow of two immiscible fluids, and (2) the equations describing the convective-dispersive transport of the dissolved contaminants. A two-dimensional, two-phase fluid flow model is used to solve the flow equations. At each time step, the two-phase flow equations are solved using an implicit scheme to solve for wetting or non-wetting phase fluid pressures, and an explicit scheme to solve for wetting phase and non-wetting phase fluid saturations. Estimates of wetting phase fluid pressures and saturations are then used as inputs to the convective-dispersion equations, which are related to the fluid flow equations by the seepage velocity. The method of characteristics, as developed in the petroleum industry, is used to solve convective-dispersive equations in a two-dimensional, saturated-unsaturated porous medium. The accuracy of the numerical simulator, when tested by comparing with available analytical solutions, other numerical solutions and experimental data, provided excellent agreement. The proposed simulator has potentially wide applications in studying the transport of gasoline hydrocarbons resulting from leakage or spills.

Introduction

Ground water contamination can result from gasoline and other petroleum products that can enter subsurface flow systems from leaking pipelines or storage tanks. Several researchers have studied ground water contamination problems associated with immiscible fluids such as fuel oil and gasoline (e.g., Williams and Wilder, 1971; McKee et al., 1972; Holzer, 1976). Since oil and gasoline are less dense that water, are immiscible in water and have their own flow potential, the theory of miscible displacement cannot be applied to ground water contamination problems associated with oil spills and leakages. A two-dimensional, two-phase fluid flow and a transport model have been developed to study the contaminant migration due to oils and gasolines in saturated and unsaturated porous media. The two-phase flow model is based on well-established techniques that were developed in the petroleum industry. The transport model is based on the method of characteristics to describe the convection and dispersion of contaminants in saturated and unsaturated porous media. The numerical simulator is tested on saturated and unsaturated flow problems for which analytical solutions, numerical solutions, and experimental data are available. A two-dimensional, saturated-unsaturated seepage problem is also solved using the method of characteristics.

Governing Equations

A two-phase fluid flow model, as developed in the petroleum industry (Breitenbach et al., 1968a, 1968b), was employed to solve the fluid flow equations. The two-phase flow equations were obtained by combining: (1) the continuity equation for each fluid phase, (2) Darcy's law for each phase, (3) a fluid conservation equation, and (4) an equation defining the capillary pressure. Using shorthand tensor notation, the continuity equations for the wetting phase fluid and the nonwetting phase fluid are, respectively:

$$\frac{\partial}{\partial x_i}(\rho_w q_{w_i}) = -\frac{\partial}{\partial t}(\rho_w \phi S_w) \qquad i=1 \text{ and } 3 \tag{1a}$$

and

$$\frac{\partial}{\partial x_i}(\rho_n q_{n_i}) = -\frac{\partial}{\partial t}(\rho_n \phi S_n) \tag{1b}$$

where w, n = subscripts for the wetting and non-wetting phase, respectively; q_i = Darcian velocity or volumetric flux per unit area in i-th direction (LT^{-1}); ρ = fluid density (ML^{-3}); ϕ = porosity of the medium; S = fluid saturation; t = time (T); x_i = two-dimensional cartesian coordinate system (L); x_1 = horizontal coordinate, and x_3 = vertical coordinate.

Darcy's Law for the wetting and non-wetting phases are, respectively:

$$q_{w_i} = -\frac{k_{x_i} k_{rw}}{\mu_w}\left(\frac{\partial P_w}{\partial x_i} + \rho_w g \frac{\partial h}{\partial x_i}\right) \tag{2a}$$

and

$$q_{n_i} = -\frac{k_{x_i} k_{rn}}{\mu_n}\left(\frac{\partial P_n}{\partial x_i} + \rho_n g \frac{\partial h}{\partial x_i}\right) \tag{2b}$$

where k_{x_i} = absolute or saturated permeability in the i-th direction (L^2); k_r = relative permeability - the ratio of effective permeability to the saturated permeability; μ = dynamic viscosity ($ML^{-1}T^{-1}$); g = acceleration of gravity (LT^{-2}); P = fluid pressure ($ML^{-1}T^{-2}$); and h = elevation head (L).

The fluid conservation equation states:

$$S_w + S_n = 1. \tag{3}$$

The capillary pressure is defined as the difference between the non-wetting phase fluid pressure and the wetting phase pressure, i.e.,

$$P_c = P_n - P_w. \tag{4}$$

The relative permeability of each fluid phase is assumed to be given by the following functional relationships:

$$k_{rw} = k_{rw}(S_w) \tag{5a}$$

and

$$k_{rn} = k_{rn}(1 - S_w). \tag{5b}$$

Following Breitenbach et al. (1968a, 1968b) and combining equations (1) through (4), we obtained the two-phase fluid flow equations. At each time step, the equations are solved using an implicit scheme to solve for wetting or non-wetting phase fluid pressures, and an explicit scheme to solve for wetting and non-wetting phase saturations. Estimates of fluid pressures and saturations are then used as inputs to the convective-dispersion equations.

Using again shorthand tensor notation, the convective-dispersion equations, for an integrated saturated-unsaturated flow domain, are (Reddell and Sunada, 1970):

$$\frac{\partial}{\partial t}\left(\rho_\alpha \phi S_\alpha C_\alpha\right) = \frac{\partial}{\partial x_i}\left[\rho_\alpha \phi S_\alpha D^*_{\alpha_{ij}} \frac{\partial C_\alpha}{\partial x_j}\right] - \frac{\partial}{\partial x_i}\left(\rho_\alpha V_{\alpha i} \phi S_\alpha\right) \qquad \begin{matrix}\alpha=w,n\\ i=1,3\end{matrix} \tag{6}$$

where $D^*_{\alpha_{ij}}$ = coefficient of hydrodynamic dispersion, a second rank tensor (L^2T^{-1}), includes mechanical dispersion and molecular diffusion; C_α = dissolved contaminant concentration in phase α (ML^{-3}); and V_{α_i} = seepage velocity (flow rate per unit pore area) of phase α in i-th direction (LT^{-1}).

Equation (6) is solved for the dissolved contaminant in each phase. The required velocity field is calculated by the two-phase flow equations. The method of characteristics (MOC) introduced by Garder et al. (1964) is used to solve (6). This technique does not introduce numerical dispersion (artificial dispersion resulting from the numerical process), and has been widely used for solving contaminant transport problems. Reddell and Sunada (1970), Pinder and Cooper (1970), Bredehoeft and Pinder (1973), and Konikow and Bredehoeft (1974) used the MOC for two-dimensional saturated flow problems, whereas Smajstrla et al. (1975) used it for one-dimensional unsaturated flow. In this study, the convective-dispersion equations are solved in an integrated saturated-unsaturated flow domain using the MOC.

The MOC algorithm is not described here in detail, since it has already been discussed in the literature (Garder et al., 1964). Briefly, in addition to the usual division of the flow region into a grid system, a set of moving points is introduced into the numerical solution. Initially, the moving points are uniformly distributed throughout the grid system with a given concentration. At each time interval, the moving points in a two-dimensional system are relocated using:

$$x_{1_\ell}^{t+\Delta t} = x_{1_\ell}^{t} + \Delta t\, V_{1_\ell}^{t+\Delta t} , \tag{7a}$$

and

$$x_{3_\ell}^{t+\Delta t} = x_{3_\ell}^{t} + \Delta t\, V_{3_\ell}^{t+\Delta t} \tag{7b}$$

where $t + \Delta t$ = new time level; t = old time level; Δt = time increment; x_{1_ℓ} and x_{3_ℓ} = coordinates of the ℓ-th moving point in the x_1 - and x_3 - directions; and V_{1_ℓ} and V_{3_ℓ} = velocities of the ℓ-th moving point in the x_1 - and x_3 - directions. When all the moving points have been relocated, each grid system is temporarily assigned a concentration, $C^{t+\Delta}$, which is the average of the concentrations $C_\ell^{t+\Delta}$ of all the moving points lying inside the grid at time $t + \Delta t$. Next, the change in concentration due to dispersion, ΔC, is calculated using an explicit, centered-in-space finite difference approximation to the first term on the right hand side of equation (6). Each moving point is then assigned a concentration according to,

$$C_\ell^{t+\Delta t} = C_\ell^{t+\Delta} + \Delta C. \tag{8}$$

To complete the step from time t to $t+\Delta t$, the concentration of the stationary grid points are calculated according to

$$C^{t+\Delta t} = C^{t+\Delta} + \Delta C. \tag{9}$$

Numerical Testing

The accuracy of the numerical simulator developed in this study was tested by comparing the numerical solutions with available analytical solutions, other numerical solutions, and experimental data. The following problems were considered: (1) a two-phase flow problem in unsaturated porous media, (2) a longitudinal dispersion problem in saturated media, (3) a longitudinal and lateral dispersion problem in saturated media; (4) a longitudinal dispersion problem in unsaturated media; and (5) a two-dimensional seepage problem in an integrated saturated-unsaturated porous media.

Two-Phase Flow In Unsaturated Media

The accuracy of the two-phase simulator was tested by comparing the numerical results against McWhorter's (1971) experimental data. McWhorter studied an one-dimensional infiltration problem in which liquid was provided at the soil surface at a constant ponding depth of 0.8 cm. The wetting phase fluid was a light hydrocarbon oil having a density of 0.7562 gcm^{-3}. Data used for the numerical simulation were: depth of model, $\ell_3 = 185$ cm, $\Delta x_3 = 5.61$ cm, saturated permeability, $k_s = 2.52 \times 10^{-8}$ cm^2, and $\phi = 0.396$. The lower boundary conditions for the one-dimensional model were zero flux conditions for both wetting (i.e., oil) and non-wetting (i.e., air) phases. The porous medium was a Poudre sand whose hydraulic properties are given in Figures 1 and 2. The imbibition cycle of the capillary pressure versus saturation curve (Figure 1) was used in the simulation. The initial condition was of uniform wetting phase saturation of 0.29.

Initially, air is at atmospheric pressure throughout the column and the liquid saturation near the surface is close to one. With infiltration, air is trapped inside the column and compressed. When the air pressure near the surface reaches a threshold value, air starts to move in a direction opposite to the movement of the liquid. However, before air can escape, the saturation near the surface must decrease to provide air permeability. Such a desaturation occurs even though there is ponding of liquid at the surface. Since the liquid pressure at the surface is constant, the increase in capillary pressure near the surface necessary to provide the air permeability must be provided by an increase in air pressure. Figure 3 shows variations of infiltration rate and air pressure with time. The infiltration rate curve in Figure 3 has a unique shape after air counterflow begins. With air counterflow, a hump was observed in the infiltration rate. However, as shown in Figure 3, the infiltration rate eventually decreases to values less than the hydraulic conductivity at $1-S_{ar}$ (S_{ar} = residual air saturation) because the air pressure buildup represents a retarding force and a reduced permeability to liquid by increasing the capillary pressure (McWhorter, 1971).

Excellent agreement was obtained between the numerical and experimental infiltration rates prior to the initiation of air counterflow (Figure 3). Air pressures predicted by the numerical model prior to air counterflow were higher than those determined experimentally. Also, there appears to be a time lag in the numerical and experimental air pressure curves prior to air counterflow.

The deviation between the numerical and experimental infiltration rate curves begins when the infiltration rate reaches its lowest value (Figure 3). At this point, air counterflow starts and the desaturation of the soil surface begins. The upward moving air front breaks through the soil surface and the compressed air escapes through the surface violently resulting in changes in medium characteristics. Contrary to this, escape of air from the medium was treated in the numerical model by a gradual desaturation of the soil surface. This particular boundary condition, immediately after initiation of air counterflow, is difficult to simulate in the numerical model. The discrepancies between the numerical and the experimental air pressure and infiltration rate curves, after the initiation of air counterflow, are believed to be due to problems in simulating such boundary conditions numerically. However, the fact that the experimental and numerical curves behave in a similar fashion provides strong evidence for the accuracy of the two-phase simulator developed in this study.

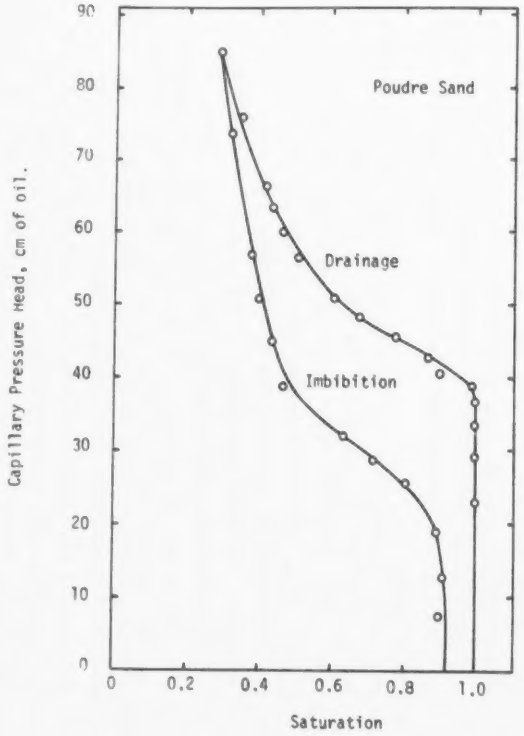

Figure 1. Capillary pressure head as a function of liquid saturation for Poudre sand.

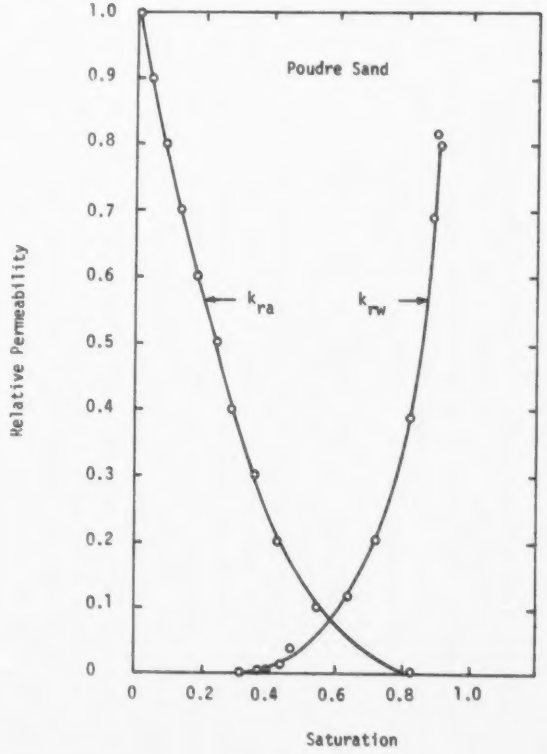

Figure 2. Relative permeability as a function of liquid saturation for Poudre sand.

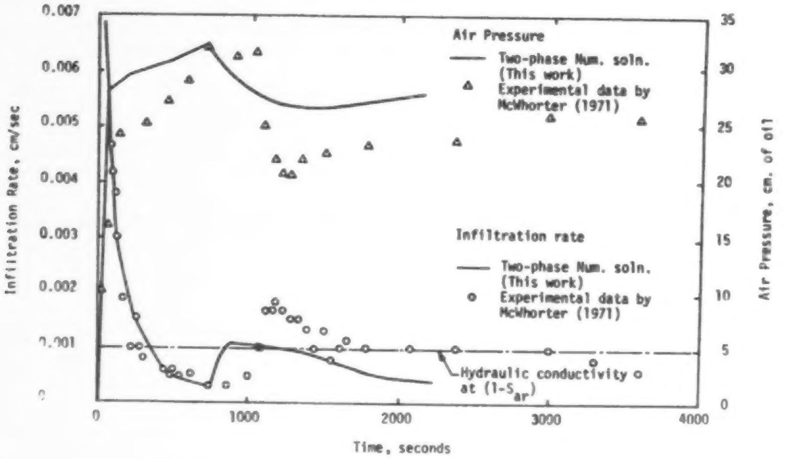

Figure 3. Comparison of experimental and numerical results for infiltration into 185-cm Poudre sand.

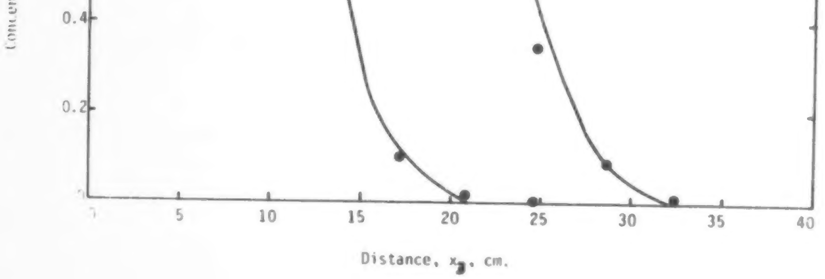

Figure 4. Comparison of analytical and numerical solutions to the longitudinal dispersion problem in saturated media.

The numerical simulation results were compared with those obtained from solution of convective-dispersion equation of the following form:

$$\frac{\partial C}{\partial t} = D_L \frac{\partial^2 C}{\partial x_3^2} - V_3 \frac{\partial C}{\partial x_3} . \tag{10}$$

The appropriate initial and boundary conditions for the problem considered are as follows:

$$\begin{aligned} C(x_3,0) &= 0 \ ; \ x_3 \geq 0 \\ C(0,t) &= C_0 ; \ t \geq 0 \\ C(\infty,t) &= 0 \ ; \ t \geq 0. \end{aligned} \tag{11}$$

Ogata and Banks (1961) used Laplace transforms to obtain the solution

$$\frac{C}{C_0} = \frac{1}{2} \left[\operatorname{erfc}\left\{ \frac{x_3 - V_3 t}{2(D_L t)^{\frac{1}{2}}} \right\} + \exp\left\{ \frac{V_3 x_3}{D_L} \right\} \operatorname{erfc}\left\{ \frac{x_3 + V_3 t}{2(D_L t)^{\frac{1}{2}}} \right\} \right] \tag{12}$$

A numerical solution was obtained using the data from Garder et al. (1964): $\Delta t = 100$ sec, $\Delta x_3 = 3.81$ cm, $D_L = 2.94 \times 10^{-3}$ cm^{2}sec^{-1}, $\phi = 0.34$, $V_3 = 0.01411$ cm sec^{-1}, $\ell_3 = 182.88$ cm, number of grids = 48, and moving points per grid = 4. The results are shown in Figure 4, and excellent agreement is indicated between numerical and analytical solutions. Reddell and Sunada (1970) made extensive studies on the number of moving points required per grid for the MOC to give accurate results. Contrary to Garder et al. (1964), they concluded that both the number and relative position of the moving points influenced the average grid concentration.

Longitudinal and Lateral Dispersion in Saturated Media

If a rectangular column ($0 \leq x_3 \leq \ell_3$, $0 \leq x_1 \leq \ell_1$) is used and a tracer source is maintained over a portion of the input area ($0 \leq x_1 \leq b$) as shown in Figure 5, then both longitudinal and lateral dispersion will occur. Assuming a homogeneous and isotropic saturated medium with unidirectional flow in the x_3 direction and $\partial C/\partial x_1 = 0$, equation (6) becomes

$$\frac{\partial C}{\partial t} = D_L \frac{\partial^2 C}{\partial {x_3}^2} + D_T \frac{\partial^2 C}{\partial {x_1}^2} - V_3 \frac{\partial C}{\partial x_3}. \tag{13}$$

The initial and boundary conditions are given by:

$$\begin{aligned} &C(x_1, 0, t) = C_0; && 0 \leq x_1 \leq b; && t \geq 0 \\ &C(x_1, 0, t) = 0; && b < x_1 < \ell_1; && t \geq 0 \\ &\frac{\partial C}{\partial x_1}(0, x_3, t) = 0, && t > 0 \\ &\frac{\partial C}{\partial x_1}(\ell_1, x_3, t) = 0, && t > 0 \end{aligned}$$

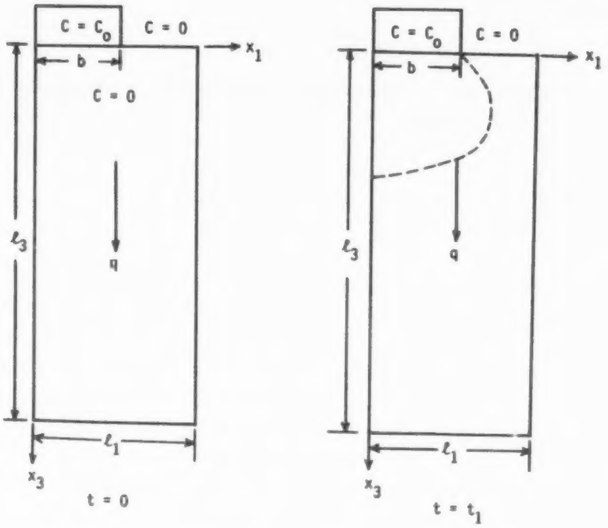
C = C$_0$
C = 0
x$_1$
b
C = 0
q
ℓ$_3$
ℓ$_1$
x$_3$
t = 0
C = C$_0$
C = 0
x$_1$
b
ℓ$_3$
q
ℓ$_1$
x$_3$
t = t$_1$

Figure 5. Schematic sketch of longitudinal and lateral dispersion problem.

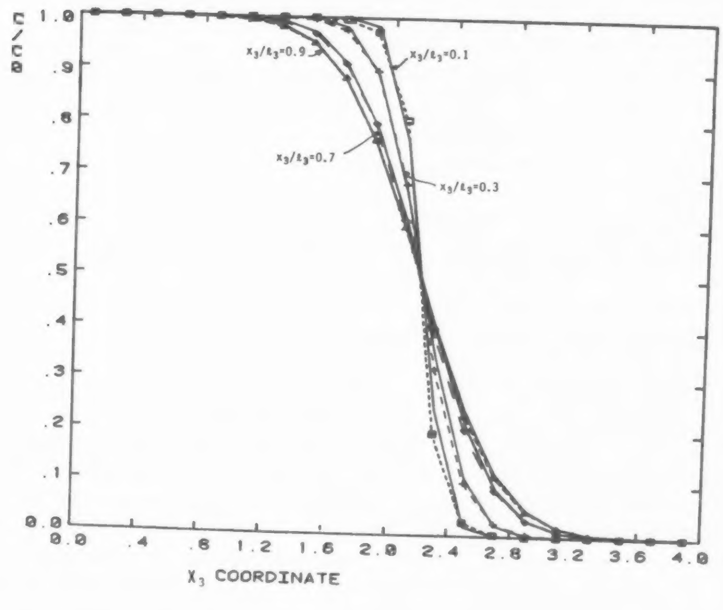

Figure 6. Comparison of numerical and approximate analytical solution for the two-dimensional dispersion problem in saturated media at steady state concentration.

$$\begin{aligned} &C(x_1, \infty, t) = \text{Bounded} \\ &C(x_1, x_3, 0) = 1, \qquad 0 \leq x_1 \leq \ell_1; \qquad x_3 > 0 . \end{aligned} \tag{14}$$

Harleman and Rumer (1963) gave the following approximate steady-state solution to equations (13) and (14):

$$\frac{C}{C_0} = \frac{1}{2} \operatorname{erfc} \left[\frac{x_1 - b}{2 \sqrt{D_T x_3 / V_3}} \right] . \tag{15}$$

The numerical simulator was tested using the following data: 25 x 20 grids on $0 \leq x_3 \leq 10$ cm and $0 \leq x_1 \leq 4$ cm, $V_3 = 0.10$ cm sec^{-1}, $D_L = 0.01$ $\text{cm}^2\text{sec}^{-1}$, $D_T = 0.001$ $\text{cm}^2\text{sec}^{-1}$, moving points per grid = 4, $\Delta x_3 = 0.4$ cm, $\Delta x_1 = 0.2$ cm, b = 2.2 cm, and $\Delta t = 2$ sec. The step input of concentration was handled numerically by letting $C/C_0 = 1.0$ for $x_1 < b$, $C/C_0 = 0.5$ for $x_1 = b$, and $C/C_0 = 0.0$ for $x_1 > b$. The same problem was considered by Reddell and Sunada (1970) to test their numerical simulator.

The results from the numerical solution of the longitudinal and lateral dispersion problem are shown in Figures 6 and 7 at t = 200 sec, and after an approximate steady state condition was achieved. For comparison, the approximate analytical solutions for the steady state case as determined from (15) are also plotted in Figures 6 and 7 as the solid lines. The accuracy of the numerical solution, although not as good as was achieved in the longitudinal dispersion problem, is acceptable. The region close to the source, i.e., $x_3 = 0$, represents the problem area where the accuracy is not so good. This occurs because of the very steep concentration in the x_1 - direction, which approaches a step function. Reddell and Sunada (1970) discussed the problem of achieving accurate numerical solutions along steep concentration profiles or when step-input functions are used. They reported that much smaller grid dimensions are necessary in these areas to obtain accurate results. It should also be noted that (15) is only an approximate analytical solution and not an exact one. Also, (15) is a steady state solution, but the numerical solutions are transient. The numerical solutions were terminated after 200 seconds of simulation and the results were changing only slightly with each additional time step. Nevertheless, the results were changing some, and a true steady state had not been achieved.

Longitudinal Dispersion in Unsaturated Media

The accuracy of our transport codes, under unsaturated flow conditions, was tested by solving the solute transport problem as described by Warrick et al. (1971). Other simulation results using this particular problem are available. For example, van Genuchten (1978) utilized Warrick et al.'s data to test his finite element program for solving solute transport under unsaturated flow conditions. The numerical results from our simulator could therefore be compared with van Genuchten's solution.

Warrick et al. obtained experimental data from a 6.1 m x 6.1 m square plot of Panoche clay loam having an approximate initial moisture content of 0.20. The soil was wetted with 7.62 cm of 0.20 N $CaCl_2$ solution, followed immediately by tracer-free water. The total infiltration time was 17.5 hours. Functional relationships for moisture content, capillary pressure, hydraulic conductivity, and specific water capacity for Panoche clay loam were given by van Genuchten (1978).

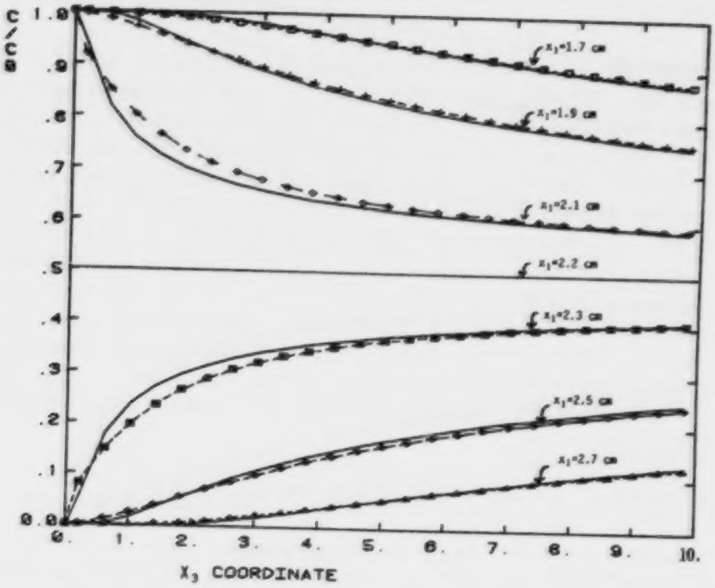

Figure 7. Comparison of longitudinal concentration distribution at steady state as calculated numerically and by an approximate analytical solution for the two-dimensional dispersion problem in saturated media.

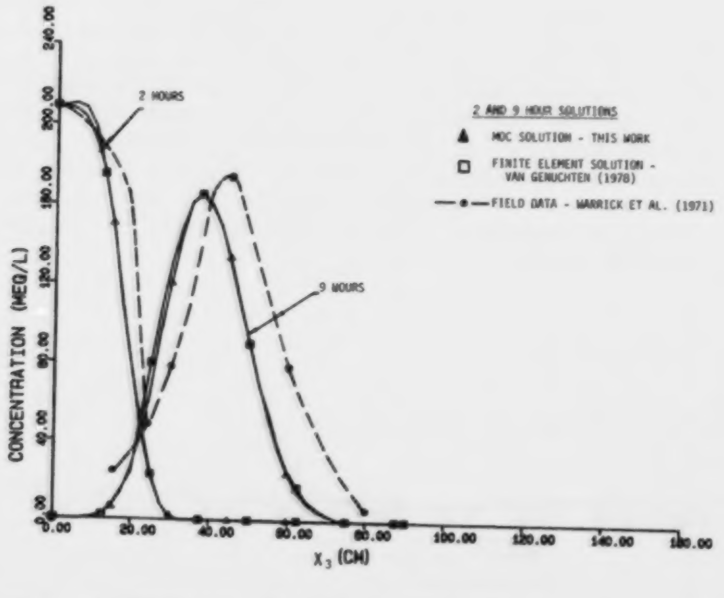

Figure 8. Comparison of experimental solute concentration profiles at 2 and 9 hours for the Panoche clay loam soil (Warrick et al., 1971) with those calculated by MOC and finite element scheme (van Genuchten, 1978).

Figure 8 is a plot of solute concentration profiles at both 2 and 9 hours using our numerical simulator. Our MOC results compared very well with those obtained by van Genuchten using his finite element scheme. Figure 8 also shows a comparison of our simulated results with field data obtained by Warrick et al. (1971). The experimental solute concentration profiles are deeper than those predicted by numerical simulation, i.e., the simulated profiles lag the measured profiles at both 2 and 9 hours; an observation also made for numerical simulations of this experiment by Warrick et al. (1971), Bresler (1973), Smajstrla et al. (1975), and Wilson and Gelhar (1981). This lag is attributed to lack of complete solute mixing within all soil pores; most of the water moved through the larger water-filled pore sequences as suggested by Warrick et al. (1971). However, this effect of immobile pore space was not taken into account in our numerical solutions.

Two-Dimensional Dispersion In Saturated-Unsaturated Media

Our numerical simulations so far either bypassed the fluid flow segments (e.g., the longitudinal and the two-dimensional dispersion problems in saturated media) or the two-dimensional dispersion segments (e.g., the two-phase fluid flow and the longitudinal dispersion problems in unsaturated media) of the simulator. The total simulator using both the two-phase fluid flow equations and the convective-dispersion equations have not been used. To test the full two-dimensional fluid flow-dispersion numerical simulator, a seepage problem was considered in which a constant source of pollution with relative concentration $C/C_0 = 1$ was infiltrated from the land surface. A schematic diagram of the problem is shown in Figure 9. The source is located between a and b (Figure 9); a constant pressure potential of $\psi_w(P_w/\rho_w g) = 0$ is maintained during the simulation. The flow domain consists of a rectangular soil slab on an impermeable base and between two parallel drains spaced at ℓ_1. The water levels in the drains on left and right are at elevations H_A and H_B, respectively and are assumed to remain constant during the simulation. The water table at time $t = 0$ is represented by cd. The distances oc, md, oa, and bm are simulated as no-flow boundaries. The lower boundary ($0 \leq x_1 \leq \ell_1$, $x_3 = \ell_3$) is also treated as a no-flow boundary. The initial condition for the tracer was $C/C_0 = 0$ for all grids.

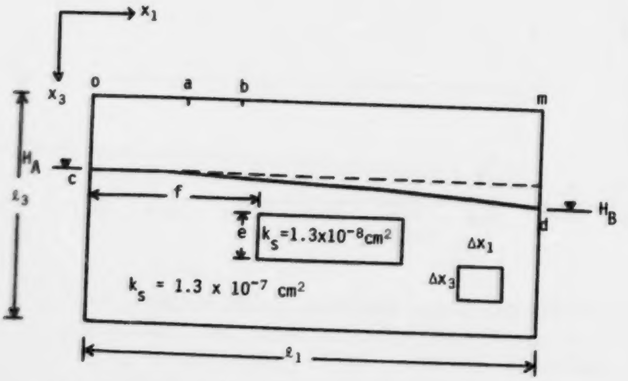

Figure 9. Schematic diagram of the two-dimensional drainage problem.

Data used for analyzing the drainage problem were: horizontal increment, $\Delta x_1 = 100$ cm; vertical increment, $\Delta x_3 = 50$ cm; $\phi = 0.3$; number of rows, NR = 13; number of columns, NC = 18; length of model, $\ell_1 =$ 1800 cm; depth of model $\ell_3 = 650$ cm; $H_A = 250$ cm; $H_B = 300$ cm; and moving points per grid = 4. The dimensions for oa and ab were 400 cm and 100 cm, respectively.

The numerical model can simulate systems with variable permeability. The saturated permeability, k_s, in the unshaded area of the model (Figure 9) was 10 times higher than that in the shaded area. The dimensions e and f (Figure 9) were 150 cm and 600 cm, respectively.

Soil hydraulic properties used in simulation are shown in Figures 10 and 11. The longitudinal and lateral dispersion coefficients, D_L and D_T, were obtained as a function of Peclet Number , P_e (Bear, 1972):

$$P_e = \frac{Vd_{50}}{D_d} \tag{16}$$

where d_{50} = mean grain size diameter (L); and D_d = molecular diffusion coefficient = 1.85×10^{-5} cm^2/s. The mean grain size diameter, d_{50}, was given by Bear (1972) as:

$$d_{50} = \left(\frac{k_s}{6.54 \times 10^{-4}}\right)^{0.5} \tag{17}$$

Following Bear (1972), equations were used to represent the functional relationships between D_L and P_e:

$$D_L/D_d = 0.72, \qquad P_e \le 0.4 \tag{18a}$$

$$D_L/D_d = 0.59 \exp\,(0.53P_e), \qquad 0.4 < P_e \le 2 \tag{18b}$$

and

$$D_L/D_d = 0.81\ P_e^{1.083} \qquad 2 < P_e \le 100\ . \tag{18c}$$

For the lateral dispersion coefficient, D_T, the following relationships were used:

$$D_T/D_d = 0.72 \qquad P_e < 1.5 \tag{19a}$$

$$D_T/D_d = 0.6\ P_e^{0.499} \qquad 1.5 < P_e \le 3 \tag{19b}$$

and

$$D_T/D_d = 0.465\ P_e^{0.746} \qquad 3 < P_e \le 100\ . \tag{19c}$$

For most situations, the contribution of molecular diffusion to hydrodynamic dispersion is negligible. Therefore, the following functional relationships for the components of the hydrodynamic dispersion tensor were assumed:

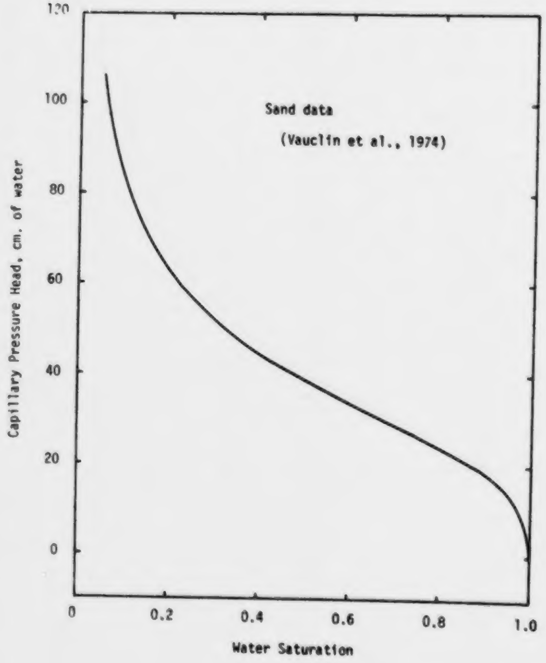

Figure 10. Capillary pressure head as a function of water saturation for the two-dimensional drainage problem.

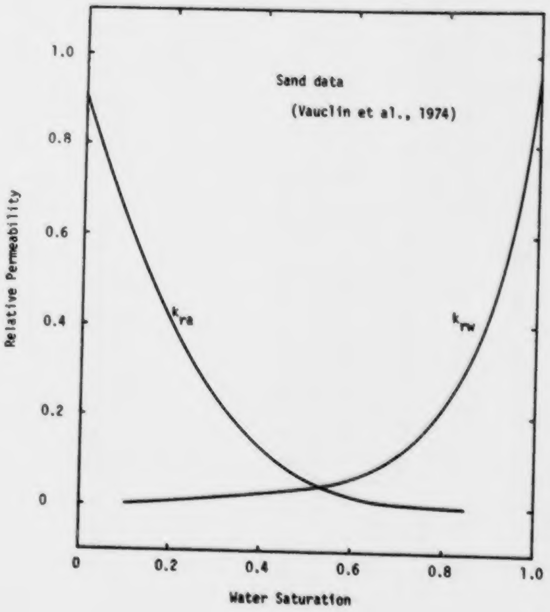

Figure 11. Relative permeabilities for water and air as a function of water saturation for the two-dimensional drainage problem.

$$D^*_{11} = D_L \frac{V_1V_1}{V^2} + D_T \frac{V_3V_3}{V^2} \tag{20a}$$

$$D^*_{33} = D_T \frac{V_1V_1}{V^2} + D_L \frac{V_3V_3}{V^2} \tag{20b}$$

$$D^*_{31} = D^*_{13} = (D_L - D_T)\frac{V_1V_3}{V^2} \tag{20c}$$

where V_1, V_3 = components of the seepage velocity (LT^{-1}), in the x_1 - and x_3 directions, respectively, and V = magnitude of the seepage velocity. The numerical simulation was continued for 100 time steps or 57.8 hours. A maximum change in water saturation of only 0.05 was allowed during any one time step. If a change in water saturation greater than 0.05 occurred, the time step size was reduced and the calculations re-done. Water pressures and water saturations were changing very slowly towards the end of the simulation. Therefore, the fluid velocities were assumed to have reached steady state. The infiltration rate of water at the land surface at steady state was 0.0234 cm/sec. The total computer time required to run the program on an AMDAHL 470 V/6 for this 13 x 18 grid network was 240 seconds of 2.4 seconds per time step.

Figure 12 shows the equipotential lines and the water table positions at 57.8 hours. The continuity of the potential field in the entire flow domain of saturated and unsaturated zones is clearly demonstrated. The water table position was obtained numerically by interpolating to zero between negative and positive water pressure heads. If the water pressures in a column were all positive, the water table position for that particular column was defined at the soil surface.

The equipotential lines of -240 cm and -250 cm in Figure 12 illustrate the effects of using a layered media. These equipotential lines are nearly vertical above the permeability transition. Within the less permeable zone, the slope of these lines changes. If the porous medium was homogeneous, these equipotential lines would have been nearly vertical through the entire depth of the medium.

To illustrate the effect of vertical and horizontal flow components, distribution of water fluxes at 57.8 hours is shown in Figure 13 using a vectorial representation. At each grid center, the vertical and horizontal seepage velocities were obtained from the Darcian flux. The resultant velocity vector was then drawn at each grid center with a length proportional to its size. As expected, the vertical component of the seepage velocity is clearly dominant in the column immediately beneath the infiltrating source. The vertical seepage velocity at the grid center immediately below the infiltrating source is 76.0 cm/hr, while the horizontal component is only 0.50 cm/hr. However, the horizontal velocity is much larger than the vertical velocity at the two outflow faces at $x_1 = 0$ and $x_1 = \ell_1$.

The effect of using a layered porous media is illustrated in Figure 13. The horizontal and the vertical components of the seepage velocity are negligible within the less permeable zone. This indicates that the majority of the flow was passing over the top of the less permeable layer. It is obvious that more flow was also taking place below the less permeable layer.

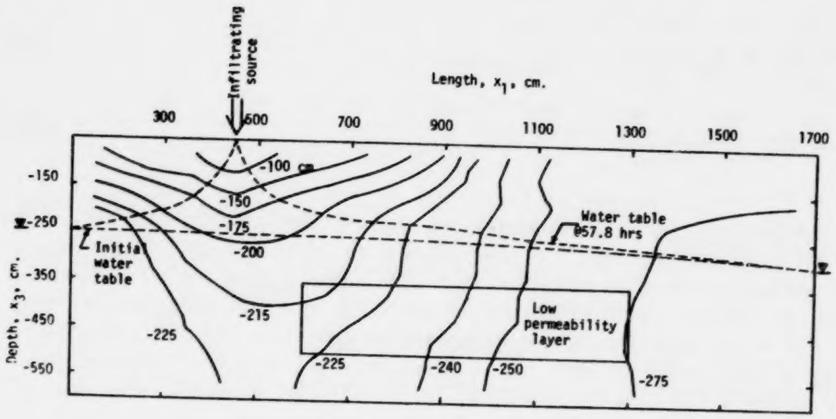

Figure 12. Equipotential map at time t = 57.8 hours.

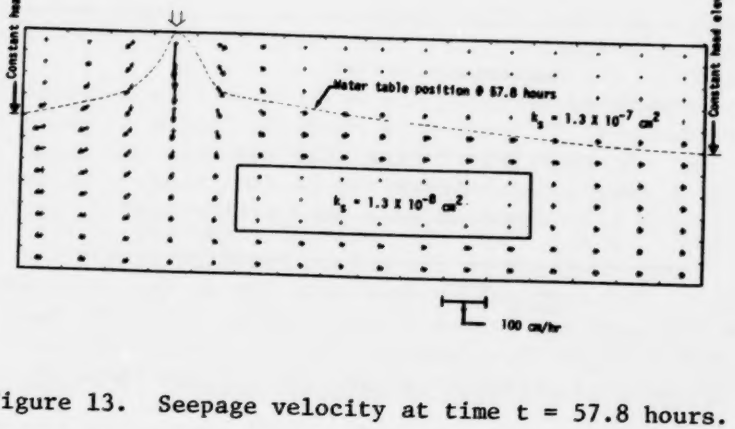

Figure 13. Seepage velocity at time t = 57.8 hours.

Figure 14 shows concentration maps of the moving points at 57.8 hours. Such a plot locates a moving point with its coordinates and concentration and when all the points are plotted, a concentration map is generated. As discussed earlier, grid concentrations are obtained by averaging the concentrations of all moving points within a grid at the end of a time step. Such an averaging scheme is often misleading, especially when the concentration of the moving points in a grid are either $C/C_0 = 1$ or $C/C_0 = 0$. A more efficient computational scheme would be one which uses the coordinates of the moving points to obtain a weighted average for the grid concentration. Reddell and Sunada (1970) developed such a computational scheme for a steady, uniform flow field. A similar scheme would be very difficult to program for the nonuniform, transient velocity field that developed in this study. In the absence of a more efficient averaging scheme, it appears desirable to plot a moving points' concentration map instead of a grid concentration map.

As the moving points exit the system at $x_1 = 0$ or $x_1 = \ell_1$, they were reintroduced at the appropriate inflow boundary. The coordinates of the reintroduced points were assigned randomly at the inflow boundary using a random number generator; it generated numbers having uniform distribution and ranging from zero to one. The reintroduced points were assigned the concentration of the appropriate inflow boundary. Also, if the number of moving points in a grid dropped below a specified minimum number, new points were introduced in that grid to bring it to a minimum number. The random number generator was again used to assign the coordinates of the new moving points and the grid or grids the fluid was emanating were taken into account in assigning the coordinates. The new points were assigned the concentration of the grid or grids the fluid was emanating. Initially, the new or old points were introduced with the same coordinates as the initial moving points in that grid. As a result, the reintroduced points traced essentially the same flow paths as the original moving points. This generated no new flow paths and no new information. These problems were corrected by reintroducing or adding points with their coordinates assigned randomly using the random number generator.

The effect of using a layered porous medium is again illustrated in Figure 14. As expected, the grids above the permeability transition zone accumulated moving points with concentrations close to $C/C_0 = 1$ and grids below the transition had concentrations close to $C/C_0 = 0$. This is due to the fact that majority of the flow is passing over the top of the less permeable layer. This points out the limitations of assuming a homogeneous and isotropic porous media which have been used in many previously developed models. The times of travel of the moving points are inversely proportional to the saturated permeability. Freeze (1972) showed that with about the same length of flow paths, the time of travel in one case was less than half of the other. This was due to the nonhomogeneous and anisotropic nature of real aquifers. The flow path with least travel time traversed high permeability layers and avoided the low gradient, low permeability layers.

Material balance errors were computed at the end of each time step for the tracer. The cumulative material balance for the tracer was defined as:

$$\begin{array}{l}\text{Cumulative tracer}\\ \text{material balance}\\ \text{error (\%)}\end{array} = \left\{ \frac{\begin{array}{c}\text{Tracer storage}\\ \text{change}\end{array} - \begin{array}{c}\text{Total tracer}\\ \text{injected}\end{array} + \begin{array}{c}\text{Total tracer}\\ \text{leaving}\end{array}}{\begin{array}{c}\text{Total tracer}\\ \text{injected}\end{array} + \begin{array}{c}\text{Total initial}\\ \text{tracer in system}\end{array}} \right\} \times 100. \qquad (21)$$

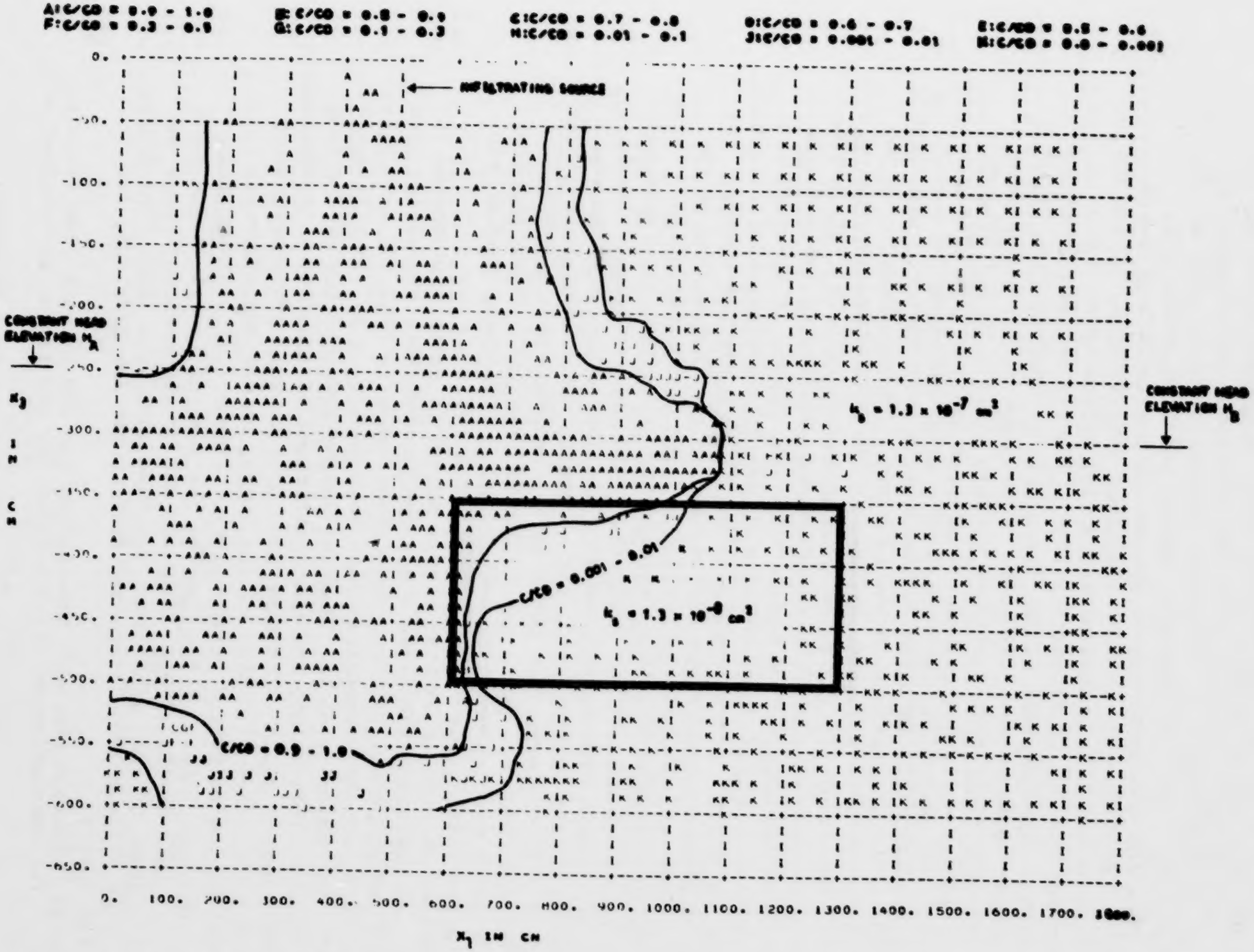

Figure 14. Moving points' concentration map at time t = 57.8 hours.

The total amount of tracer injected into the system was calculated by integrating the infiltrated water volume multiplied by the relative concentration, C/C_0, of the infiltrating tracer. The above integration was carried out over the appropriate inflow boundaries at $x_3 = 0$. The total amount of tracer leaving the system was calculated by integrating the water volume leaving the system over the outflow boundaries at $x_1' = 0$ and $x_1 = \ell_1$ and multiplying it by the relative concentration, C/C_0, of the outgoing tracer. The cumulative amount of tracer in the system at any time was calculated by summing the average grid concentration over all grid nodes in the system. The change in tracer storage was defined as the difference between the cumulative and initial tracer storage. Differential material balance error was also computed for each time step. The expression for differential error was analogous to that of cumulative error.

The material balance error for the tracer is very much dependent on the numerical scheme of MOC. In the first few minutes of simulation, no moving points with the concentration of $C/C_0 = 1$ have moved into the system. However, the amount of tracer injected into the system was calculated by integrating the infiltrated water volume multipled by its tracer concentration of $C/C_0 = 1$. Since in the initial few minutes of simulation, the tracer storage change and the tracer leaving the system are essentially zero, the cumulative error, using (21), is calculated as 100 percent. But as time progresses, moving points move into the system and the material balance error begins to decrease. The cumulative tracer material balance error near the end of 57.8 hours of simulation was on the order of 2.0 to 2.5 percent. Much better results could be obtained if the cumulative tracer storage was calculated based on the area occupied by the isochlors (Figure 14) instead of using the average grid concentration. Differential tracer material balance errors ran about 5 to 10 percent near the end of simulation. Because of the nature of MOC, it was difficult to keep a check on the differential tracer material balance errors. However, the cumulative tracer errors obtained near the end of simulation were considered to be very reasonable.

Conclusions

A two-dimensional, two-phase numerical model has been developed to describe the flow and transport of organic contaminants resulting from gasoline spills and leakage. The two-phase flow model was tested by comparing the numerical results with the available experimental data (McWhorter, 1971) for an one-dimensional infiltration problem. The upper boundary condition was a constant ponding depth of 0.8 cm of light hydrocarbon oil. Qualitatively, both the numerical and experimental infiltration rates and air pressure curves behaved in a similar manner before and after air counterflow started. The deviation between the numerical solution and experimental data, after air counterflow started, was believed to be caused by change in medium characteristics.

A longitudinal dispersion problem under steady saturated flow, a two-dimensional dispersion problem under steady saturated vertical flow, and a longitudinal dispersion problem under transient unsaturated flow conditions were solved to test the transport portion of the simulator. The accuracy was tested by comparison with available analytical solutions, other numerical solutions, and experimental data. Comparisons provided excellent agreement in all cases except for the field experimental data of Warrick et al. (1971). However, in view of nonhomogeneous field conditions, comparison of our simulated results with experimental data are considered to be very good. The use of the method of characteristics (MOC) effectively eliminated numerical dispersion in the simulation of miscible displacement in both saturated and unsaturated soils. In contrast, several investigators have shown that in certain convection-dominated flow systems, the Galerkin finite element formulation will produce excessive dispersion and/or oscillation even if higher order elements are used (Zienkiewicz, 1977) 20

A typical two-dimensional drainage problem was solved in a nonhomogeneous, integrated saturated-unsaturated medium using the MOC. A variety of outputs, such as an equipotential map or a moving points' concentration map showing isochlors, were obtained at selected time steps. The limitations of the assumption of a homogeneous medium were illustrated by the accumulation of moving points at a transition from a higher to a lower permeability. The numerical simulator can be applied to ground water contamination problems due to gasoline spills and leakage, provided values of the inputs such as relative permeabilities and capillary pressures, saturations of immiscible contaminants, and field estimates of dispersivities are known.

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UNCERTAINTY IN GROUND WATER TRANSPORT MODELING

by

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ABSTRACT

A variety of computer codes are available to model the transport of chemical constituents in ground water. The major drawback of all of the codes, however, is the uncertainty of the input parameters. Most codes require input of finite values and the output is a finite concentration at some point in space and time. Hydrogeologists are well aware that aquifer coefficients usually range widely even within relatively short distances. Moreover, the data with which we work also must be taken with some degree of caution, such as well or test hole logs and samples. In most cases, an evaluation of the data will permit us to generalize to the extent that, with some degree of confidence, we can assume that a value will range within certain limits. For example, with a given set of circumstances we might assume that groundwater velocity would probably average about 1.5 feet/day, but because of the range in grain size it might be as low as .5 or as high as 2.5. Thus we have introduced a degree of uncertainty.

Our approach to the information problem is to utilize a two dimensional solute transport model to which has been added a Monte Carlo routine. Monte Carlo simulations can provide a useful means of incorporating the uncertainty of input parameters into deterministic solute transport models. It is a "brute force" approach. Our stochastic model provides a number of statistical outputs than can be interpreted in more familiar terms, such as risk analysis. Moreover, the code has been designed for use on microcomputers; it is written in FORTRAN and is menu driven.

As an example, consider the well known case of chromium

contamination on Long Island. Let us assume that the saturated thickness ranges from 105 to 115 feet, effective porosity from .34 to .36, interstitial velocity from 1.45 to 1.55 feet/day, longitudinal and transverse dispersivity from 69 to 71 feet and 13 to 15 feet, respectively, retardation of 1, and source loading from 51 to 53 lb/day. Assume also an observation point 4200 feet directly downgradient from the lagoons, which have been operating for 2800 days. After 1024 iterations output indicates that the concentration of chromium at the observation point could range betwen 7.2 and 9.7 mg/l, but it has the most probable concentration of 8.4 mg/l. Output also indicates that there is a 90% certainty (risk) that the concentration will be equal to or less than 9.0 mg/l and a 10% certainty that it will be equal to or less than 7.8 mg/l.

Presently the model calculates the concentration only at one specific site for each simulation. We are in the process of developing a program that will generate sufficient data to provide a map. The map will show the most probable concentration at each point or it will be based on a user-designated risk value. The latter might show, for example, the concentration distribution that 90% of the time would be equal to or less than the values listed. This model will be limited to mainframe use.

INTRODUCTION

A variety of computer codes are available to model the transport of chemical constituents in ground water. The major drawback of all of the codes, however, is the uncertainty of the the data (input parameters) that the model requires, such as effective porosity, aquifer thickness, and so on. Most codes require input of finite values and the output is a finite concentration at some point in space and time. Hydrogeologists are well aware, however, that aquifer coefficients usually range widely, both vertically and horizontally, even within relatively short distances. Moreover, the information with which we work, such as well or test hole logs and samples, also must be taken with some degree of uncertainty and caution because they represent only an overall and simplified condition. In most cases, an evaluation of the data will permit us to generalize to the extent that, with some degree of confidence, we can assume that a value will range within certain limits. For example, with a given set of circumstances we might assume that ground-water velocity at a particular site would probably average about 1.5 feet per day, but because of the known or assumed range in grain size or gradient, it might be as low as .5 or as high as 2.5 feet per day. Thus, we have introduced a certain amount of realism to our problem, but this has also resulted in the addition of a degree of uncertainty into our data base.

In addition to the questionability of aquifer and geochemical parameters that are required for model use, there is also some uncertainty as to the proper choice of the flow and solute transport equations on which the model is based. The model uncertainty arises from the many assumptions that must be made to keep the analysis manageable. Each assumption affects the results of the model to an undetermined, but hopefully small, extent. Examples include the use of Darcy's law, even when it is only marginally acceptable; the use of linear partial differential equations, even when the actual behavior is nonlinear; and the use of a potential flow model for hydrodynamic dispersion phenomena, which may be attributed to several physical processes occurring simultaneously. Although exceedingly important, model uncertainty is not herein considered. Rather, an appropriate mathematical model is assumed to be available that adequately describes the solute transport process. With this assumption, the future concentration distributions can be determined and predicted provided that the initial and boundary conditions, and the physical parameters describing the aquifer, i.e. the model parameters, can be specified exactly.

Our approach to the information problem is to utilize a two dimensional solute transport model to which has been added a Monte Carlo routine. Monte Carlo simulations can provide a useful means of incorporating the uncertainty of input parameters into deterministic solute transport models. Our stochastic model provides a number of statistical outputs that can be interpreted in more familiar terms, such as risk analysis. Moreover, the code has been designed for use on microcomputers; it is written in both FORTRAN and BASIC and is menu-driven.

The major advantages of this statistical approach are particularily evident when one considers regulatory permitting activities and legal actions. Imagine sitting on the witness stand after presenting your model study and interpretations to a bored judge, a confused jury, and a hostile opposing attorney. In your study you used poorly measured or assumed values (as we nearly always do) and yet the model predicted that an exact concentration would occur at some specified point at some designated time in the future. The attorney for the opposition might say "You assumed a velocity of 1.5 feet per day. Could it have been 1.3 or 1.7?" The answer, of course, is "Yes". "Would these different values have changed the concentration that your model predicted?" "Yes". Similar questioning would likely follow incorporating each of the input parameters and the closing remark would probably make some reference to a supposed scientist who hasn't the slightest idea of what he/she is doing, but is merely using some magical black box, called a computer, to provide credibility to an analysis that was based totally on assumptions and speculation. Therefore, all of the testimony should be ignored.

On the other hand, with the model described herein input parameters are characterized by a range of values--an estimated minimum, an estimated maximum, and the expected. The output allows the investigator to examine the probability of occurrance of a concentration at a point in space and time. It removes, or at least makes more managable, the uncertainly of the data base. Furthermore, it weakens the basis for cross examination.

PARAMETER UNCERTAINTY AND STOCHASTIC MODELS

Generally speaking, model input parameters are estimated from a few discrete observations and assumptions and are seldom, if ever, completely determined. Even if it is assumed that the data set contains neither measurement nor analytical errors, the estimates of system variables would still be uncertain to some extent. A more reasonable approach, perhaps, is to define a range of possible values, each having a probability of realization. In other words, each input variable is represented by a relative frequency distribution (probability density function) rather than a single estimated value. The deterministic model is equally applicable to any value in this range and, therefore, will define a corresponding ensemble with corresponding probabilities. The equations describing the behavior of the system become equations for random functions or stochastic differential equations. In addition to following the physical laws of consersvation of mass, Darcy's law, and physical/chemical reactions, the stochastic differential equations for flow or solute transport take into account the variable nature of the system. Since system randomness can be taken into consideration explicitly in such models, these models, sufficiently developed, may lead to more realistic decision making.

Stochastic models can evolve through two main approaches. One approach that has been used to describe flow and dispersion processes in porous media is to directly employ probabilistic concepts to small subdivisions of the system (Bakr, et al., 1978; Chirlin and Dagan, 1980; Dagan, 1979, 1981, 1982a, 1982b; Delhomme, 1979; Dettinger and Wilson, 1981; Gelhar, et al., 1979; Gutjahr, et al., 1978; Neuman and Yakowitz, 1979; Neuman, et al., 1980; Neuman, 1980; Prickett, et al., 1981; Sager, 1978, 1979; Scheidegger, 1954; Simmons, 1982; Smith and Schwartz, 1980, 1981; Tang and Pinder, 1979). This approach is very attractive since the solution directly yields the parameters of the probabilty density functions describing the solution space. These models are comparatively easy to formulate, but their solution entails currently unsolved mathematical problems (Sager, 1978). The theory of linear partial differential equations and nonconstant coefficients is far from complete, and for random coefficients the situation is even worse (Frisch, 1968). The principal difficulty arises from the fact first observed by Kraichman (1962) that despite its apparent linearity, the equation is nonlinear in stochastic quantities. Since the solution of a linear equation depends nonlinearly upon the coefficients. Only relatively simple problems are solvable.

Another approach to the development of a stochastic model for fluid flow or solute transport in aquifers is the application of Monte Carlo simulations. This approach has been used in the design and optimization of chemical processes (Al-Zakri and Bell, 1981; Berryman and Himmelblau, 1971, 1973; Buckley, 1950; Dittmar and Hartmann, 1976; Grossman and Sargent, 1978; Lashmet and Szczepanski, 1974; Lin and Slattery, 1982; Takeichiro, et al., 1970; Weisman and Holzman, 1972; Wen and Chang, 1968) and to a lesser extent in groundwater flow (Cooley, 1977, 1979; Rao et al., 1981; Smith and Freeze, 1979a, 1979b; Tang and Pinder, 1979). Knowing the relative frequency distribution for each variable in the deterministic model, random values selected from these distributions are substituted for these variables. By repeatedly sampling the input parameter distributions and solving the deterministic model, the frequency distribution of the model response can be generated. This approach can be likened to taking a bean, representing a variable, from each jar of a group (hydrologic parameters), solving the equation and then, randomly, taking a bean from each of the jars again in order to resolve the equation. Each solution then forms a new "bean", all of which are placed in another jar and later used for statistical analysis.

Monte Carlo simulations represent a brute force approach if compared to the direct solution of a stochastic partial differential equation. On the other hand, Monte Carlo methods avoid the mathematical problems associated with nonconstant coefficients and can be applied to nonlinear as well as linear systems. The main disadvantage is that large sample sizes may be required to adequately define the relative frequency distribution. Each sample requires the solution of the deterministic model. Neither approach is more correct than the other, since the models are only approximations in any case. However, in both approaches the introduction of randomness into the model should reflect, to the extent possible, what is happening physically in the process.

The application of a Monte Carlo simulation to a two-dimensional solute transport model is described herein. Our intent is to demonstrate one approach to introducing parameter uncertainty into solute transport models and the effect of this one type of uncertainty on model results. This example is somewhat oversimplified in that autocorrelation between aquifer properties, such as porosity and permeability, and correlation of forcing functions, such as source rates and/or concentration, with time have been neglected. However, correlation of input parameters may be very important in the correct interpretation of many stochastic models.

EXAMPLE OF MONTE CARLO SIMULATION

An example problem is based on the well known chromium contamination case in southeastern Nassau County, New York. The hydrogeology and history of contamination documented by Perlmutter and Lieber (1970) are briefly summarized.

The general area lies in southeastern Nassau County, Long Island, New York, at an industrial park at South Farmingdale (Figure 1). During World War II, the industrial park was occupied by an aircraft company whose cadmium and chromium enriched metal plating waste was the source of much of the heavy-metal contamination in a shallow glacial aquifer.

The contaminated upper glacial aquifer extends from the water table (0 to 15 feet below the ground surface) to the top of the Magothy aquifer, which lies 80 to 140 feet below the ground surface. The upper unit consists largely of fine to coarse sand and gravel. In some parts of the aquifer, thin lenses of fine to medium sand, as well as some silt, are interbedded with the coarse material. Data from scattered well borings indicates that the lower 8 to 10 feet of the upper unit may consist of silty and sandy clay.

The principal direction of ground-water flow in the glacial aquifer is from north to south and the regional hydraulic gradient is approximately 0.0025 ft/ft. The average coefficient of permeability is assumed to be 1,600 gal/day/ft^2, while the porosity averages 0.35 (Perlmutter and Lieber, 1970). The ground-water movement is horizontal throughout the area of interest, with the exception of local recharge and discharge areas where vertical or oblique flow may be predominant.

The chromium contamination in the upper glacial unit was derived from the disposal of metal-plating and anodizing waste water to unlined basins. During World War II, and for several years thereafter, essentially untreated plating-water effluent was recharged to the aquifer through these disposal basins. Perlmutter and Lieber (1970) estimated that during the early 1940's as much as 200,000 to 300,000 gallons of effluent containing approximately 52 pounds of chromium was discharged daily into the glacial unit. At the end of World War II, the amount of plating-waste effluent was decreased substantially and a chromium removal unit was installed in 1949.

By 1949, or approximately 9 years after the start of plating-waste disposal, a plume of contaminated water had migrated southward approximately 3,900 feet with a maximum width of about 850 feet. Maximum observed chromium concentration was approximately 40 mg/l. Since 1949 the chromium concentrations have decreased substantially in nearly

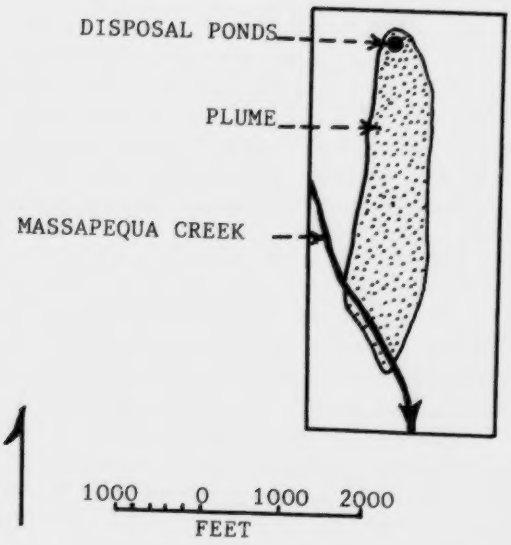
DISPOSAL PONDS
PLUME
MASSAPEQUA CREEK
1000 0 1000 2000
FEET

Figure 1. Sketch maps of the leachate plume at South Farmingdale, N.Y.

all sections of the plume. The maximum observed concentration was about 10 mg/l in 1962. However, the plume of contaminated water had migrated farther southward beyond Massapequa Creek, a small stream that serves as a natural drain for part of the contaminated ground water. The plume was approximately 4,300 feet long with a maximum width of almost 1000 feet in 1962.

Analytical Model

The governing differential equation, which is assumed to describe the conservation of mass in this situation, can be written as

$$R_d \frac{\partial C}{\partial t} + V^* \frac{\partial C}{\partial x} = (\alpha_x V^*) \frac{\partial^2 C}{\partial_x^2} + (\alpha_y V^*) \frac{\partial^2 C}{\partial_y^2} - R_d \lambda C$$

where

C = concentration of tracer in solution,
R_d = retardation coefficient,
t = time
V^* = average intersitital velocity,
x = distance in the direction of ground-water flow,
y = horizontal distance normal to the direction of ground-water flow,
α_x = longitudinal dispersivity,
α_y = transverse dispersivity,
λ = first-order decay constant.

A solution to Equation 1 is (Hunt, 1978; Wilson and Miller, 1978):

$$C = \frac{C_o Q'}{4\pi\theta \, (D_x^* D_y^*)^{0.5}} \, \mathrm{EXP} \, \left(\frac{V_x^*}{2D_x^*}\right) W(U,B)$$

where θ is the effective porosity of the aquifer, Q' is the volumetric source rate per unit depth of aquifer with a tracer concentration of C_o, and $D^* = \quad V^*$ is a dispersion coefficient. The term $W(U,B)$ is commonly referred to as the "well function for leaky artesian aquifers" (Hantush, 1956) with

$$U = \frac{\left(\frac{V_x^*}{D_x^*}\right)^2 + \frac{D_x^*}{D_z^*}\left(\frac{V_y^*}{D_x^*}\right)^2}{\frac{4V^2 t}{R_d D_x^*}} \qquad B = \frac{1}{2}\left[\left(\frac{V_x^*}{D_x^*}\right)^2 + \frac{D_x^*}{D_y^*}\left(\frac{V_y^*}{D_x^*}\right)^2\right]^{1/2}\left[1 + \frac{4D_x^* R_d \lambda}{V^{*2}}\right]^{1/2}$$

Input Parameters

In their report, Perlmutter and Lieber (1970) listed the ranges and average of many, but not all, of the aquifer and loading parameters. Ground-water velocity, for example, reportedly ranged from a low of .5 ft/day (local) to a high of 2 ft/day, with an average between 1 and 2 ft/day. Assuming an aquifer thickness that ranges from 80 to 140 feet and the lower 8 to 10 feet consisting of silty and sandy clay, the actual zone of potential mixing could be as little as 70 feet or as great as 132 feet. Laboratory analyses of porosity ranged from 34.3 to 40.7 percent. The longitudinal and transverse dispersion coefficients are the most difficult parameters to estimate. Based on the results of a numerical model for the same site, Pinder (1973) estimated longitudinal and transverse dispersivities of 69.9 and 14.0 feet, respectively.

Chromium is believed to be a conservative material in this system (Perlmutter and Lieber, 1970) and both adsorption and chemical/biological decay can be neglected. Therefore, the contaminant will not be retarded and will flow at the same velocity as the water ($R_d = 1$).

A steady mass injection rate of 52 lb/day of chromium was assumed from the time of initial injection to mid-1949, or approximately 2800 days. At this time, the actual extent of the contamination was estimated, and the chromium removal unit was installed. Actually the chromium concentration in the waste stream ranged from 28 to 29 mg/l and the estimated discharge to the disposal basins ranged from 200,000 to 300,000 gallons per day. These data convert to a minimum loading of 47 lb/day and a maximum of 72.

The model input data for this example problem are summarized in Table 1. Since this program is menu driven, examples of monitor display and operator entered values are shown in Figure 2. Triangular probability density functions were chosen for ease of visualization in this example, with upper and lower limits being selected from Perlmutter and Lieber's report.

The average values of the model parameters were used in the deterministic model to calculate the concentration of the plume at Massapequa Creek (x = 4200 ft, y = 0 ft) after 2800 days of plating-water recharge to the aquifer from the disposal basins. The calculated concentration is 8.34 mg/l at this time and distance from the source. This single value represents the results of a deterministic modeling approach.

A Monte Carlo simulation was then carried out using the same data. The frequency distributions of the calculated concentration with a sample size of 128 is presented in Figures 3 and 4. The relative frequency histogram for concentration is approaching a normal distribution, but as shown in Figure 5, the mean and standard deviation is changing with sample size.

Relative frequency histograms representing a sample size of 2048 are shown in Figures 6 and 7. Note that the frequency distribution is much smoother. As shown in Figure 8, the mean and standard deviation of this distribution have become virtually independent of sample size. Thus, the sample

PARAMETER	LOWER LIMIT	EXPECTED	UPPER LIMIT
X OBSERVATION POINT (FT)	4200.	4200.	4200.
Y OBSERVATION POINT (FT)	0000.	0000.	0000.
TIME OF INTEREST (DAYS)	2000.	2000.	2000.
INTERSTITIAL VEL. (FT/DY)	1.0	1.5	2.0
LOADING RATE (LB/DY)	47.0	52.0	72.0
POROSITY	.343	.350	.407
X DISPERSIVITY (FT)	60.0	70.0	72.0
Y DISPERSIVITY (FT)	13.0	14.0	15.0
HALF LIFE (YEARS)	2000.	2000.	2000.
AQUIFER THICKNESS (FT)	70.0	110.0	132.0
RETARDATION COEFFICIENT	1.0	1.0	1.0

Table 1. Input data for analytical solute-transport model.

```
 FOR INTERSTITIAL VEL. (FT/DY):
 ENTER LOWER LIMIT?
.5


 ENTER EXPECTED   ?
1.5

 ENTER UPPER LIMIT?
2

 ENTER TYPE OF DISTRIBUTION:
     1.  UNIFORM
     2.  TRIANGLER
 ENTER TYPE?
2

 FOR LOADING RATE (LB/DY)    :
 ENTER LOWER LIMIT?
47

 ENTER EXPECTED   ?
52

 ENTER UPPER LIMIT?
72

 ENTER TYPE OF DISTRIBUTION:
     1.  UNIFORM
     2.  TRIANGLER
 ENTER TYPE?
2

 FOR POROSITY              :
 ENTER LOWER LIMIT?
.343

 ENTER EXPECTED   ?
.35

 ENTER UPPER LIMIT?
.407

 ENTER TYPE OF DISTRIBUTION:
     1.  UNIFORM
     2.  TRIANGLER
 ENTER TYPE:
2
```

```
 FOR X DISPERSIVITY (FT)
 ENTER LOWER LIMIT?
68


 ENTER EXPECTED   ?
70

 ENTER UPPER LIMIT?
72

 ENTER TYPE OF DISTRIBUTION?
     1.  UNIFORM
     2.  TRIANGLER
 ENTER TYPE?
2

 FOR Y DISPERSIVITY (FT)    :
 ENTER LOWER LIMIT?
13

 ENTER EXPECTED?
14

 ENTER UPPER LIMIT?
15

 ENTER TYPE OF DISTRIBUTION:
     1.  UNIFORM
     2.  TRIANGLER
 ENTER TYPE?
2

 FOR AQUIFER THICKNESS (FT)    :
 ENTER LOWER LIMIT:
70

 ENTER EXPECTED   ?
110

 ENTER UPPER LIMIT?
132

 ENTER TYPE OF DISTRIBUTION:
     1.  UNIFORM
     2.  TRIANGLER
 ENTER TYPE?
2
```

Figure 2. Listing of input of interactive program. User enters boldface numbers in response to questions.

```
                      UNCERTAINTY MODEL
          --POLLUTANT CONCENTRATION   FOR A LEACHATE PLUME
          AFTER       128ITERATIONS


NUMBER OF VALUES FOR CONCENCENTRATION (MG/L)        128
MINIMUM CONCENCENTRATION (MG/L)             .967340E-01
MAXIMUM CONCENCENTRATION (MG/L)             .220549E+02
CATEGORY            RANGE              COUNT     PERCENTAGES
 NUMBER     LOW VALUE,    HIGH VALUE          BELOW  INSIDE  ABOVE
---------------------------------------------------------------
BELOW                     .967340E-01    0              .00 100.00
     1      .967340E-01   .119464E+01    4      .00    3.13  96.87
     2      .119464E+01   .229255E+01    2     3.13    1.56  95.31
     3      .229255E+01   .339046E+01    3     4.69    2.34  92.97
     4      .339046E+01   .448837E+01    6     7.03    4.69  88.28
     5      .448837E+01   .558627E+01    6    11.72    4.69  83.59
     6      .558627E+01   .668418E+01   14    16.41   10.94  72.66
     7      .668418E+01   .778209E+01   17    27.34   13.28  59.38
     8      .778209E+01   .888000E+01   15    40.63   11.72  47.66
     9      .888000E+01   .997791E+01   14    52.34   10.94  36.72
    10      .997791E+01   .110758E+02   16    63.28   12.50  24.22
    11      .110758E+02   .121737E+02   17    75.78   13.28  10.94
    12      .121737E+02   .132716E+02    6    89.06    4.69   6.25
    13      .132716E+02   .143695E+02    6    93.75    4.69   1.56
    14      .143695E+02   .154674E+02    1    98.44     .78    .78
    15      .154674E+02   .165654E+02    0    99.22     .00    .78
    16      .165654E+02   .176633E+02    0    99.22     .00    .78
    17      .176633E+02   .187612E+02    0    99.22     .00    .78
    18      .187612E+02   .198591E+02    0    99.22     .00    .78
    19      .198591E+02   .209570E+02    1    99.22     .78    .00
    20      .209570E+02   .220549E+02    0   100.00     .00    .00
ABOVE       .220549E+02                  0   100.00     .00
```

Figure 3. Frequency distribution of chromium concentration after 128 iterations.

```
             UNCERTAINTY MODEL
--POLLUTANT CONCENTRATION   FOR A LEACHATE PLUME
AFTER      128ITERATIONS


          RELATIVE FREQUENCY HISTOGRAM FOR VALUES
PERCENT   WITHIN EACH CATEGORY
 13.28+              *       *
      I              *     * *
      I              * *   * *
      I            * * * * * *
      I            * * * * * *
  8.85+            * * * * * *
      I            * * * * * *
      I            * * * * * *
      I            * * * * * *
      I            * * * * * *
  4.43+        * * * * * * * * * *
      I  *     * * * * * * * * * *
      I  *   * * * * * * * * * * *
      I  * * * * * * * * * * * * *
      I  * * * * * * * * * * * * * *         *
   .00+* * * * * * * * * * * * * * * * * * * * * *
       --------------------------------------------+
       BL 1 2 3 4 5 6 7 8 91011121314151617181920AB
                  CATEGORIES
```

Figure 4. Relative frequency histogram after 128 iterations.

```
              UNCERTAINTY MODEL
--POLLUTANT CONCENTRATION   FOR A LEACHATE PLUME
AFTER       128ITERATIONS
```

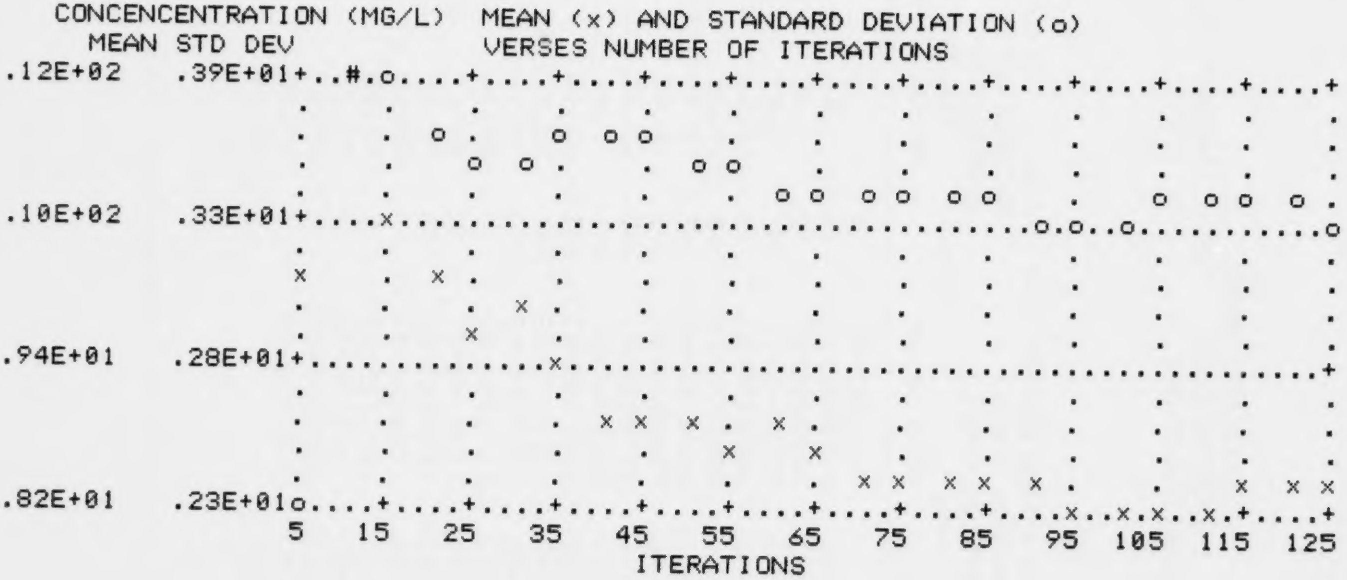

Figure 5. Mean and standard deviation of chromium concentration distribution as a function of sample size (after 128 iterations).

```
                     UNCERTAINTY MODEL
           --POLLUTANT CONCENTRATION   FOR A LEACHATE PLUME
           AFTER     2048ITERATIONS


NUMBER OF VALUES FOR CONCENCENTRATION (MG/L)     2048
MINIMUM CONCENCENTRATION (MG/L)           .967340E-01
MAXIMUM CONCENCENTRATION (MG/L)           .220549E+02
CATEGORY            RANGE                COUNT      PERCENTAGES
 NUMBER     LOW VALUE,    HIGH VALUE              BELOW  INSIDE   ABOVE
---------------------------------------------------------------------
BELOW                     .967340E-01      0                .00  100.00
    1       .967340E-01   .119464E+01     59        .00    2.88   97.12
    2       .119464E+01   .229255E+01     60       2.88    2.93   94.19
    3       .229255E+01   .339046E+01     87       5.81    4.25   89.94
    4       .339046E+01   .448837E+01     88      10.06    4.30   85.64
    5       .448837E+01   .558627E+01    114      14.36    5.57   80.08
    6       .558627E+01   .668418E+01    167      19.92    8.15   71.92
    7       .668418E+01   .778209E+01    196      28.08    9.57   62.35
    8       .778209E+01   .888000E+01    205      37.65   10.01   52.34
    9       .888000E+01   .997791E+01    261      47.66   12.74   39.60
   10       .997791E+01   .110758E+02    238      60.40   11.62   27.98
   11       .110758E+02   .121737E+02    207      72.02   10.11   17.87
   12       .121737E+02   .132716E+02    153      82.13    7.47   10.40
   13       .132716E+02   .143695E+02     94      89.60    4.59    5.81
   14       .143695E+02   .154674E+02     53      94.19    2.59    3.22
   15       .154674E+02   .165654E+02     38      96.78    1.86    1.37
   16       .165654E+02   .176633E+02     14      98.63     .68     .68
   17       .176633E+02   .187612E+02      5      99.32     .24     .44
   18       .187612E+02   .198591E+02      6      99.56     .29     .15
   19       .198591E+02   .209570E+02      2      99.85     .10     .05
   20       .209570E+02   .220549E+02      1      99.95     .05     .00
ABOVE       .220549E+02                    0     100.00     .00
```

Figure 6. Frequency distribution of chromium concentration after 2048 iterations.

```
                    UNCERTAINTY MODEL
       --POLLUTANT CONCENTRATION   FOR A LEACHATE PLUME
       AFTER     2048ITERATIONS


         RELATIVE FREQUENCY HISTOGRAM FOR VALUES
PERCENT  WITHIN EACH CATEGORY
  12.74+                  *
       I                  * *
       I                  * *
       I                * * * *
       I              * * * * *
   8.50+            * * * * * *
       I            * * * * * * *
       I            * * * * * * *
       I          * * * * * * * *
       I          * * * * * * * *
   4.25+      * * * * * * * * * * *
       I      * * * * * * * * * * *
       I  * * * * * * * * * * * * * *
       I  * * * * * * * * * * * * * * *
       I  * * * * * * * * * * * * * * * *
    .00+* * * * * * * * * * * * * * * * * * * * * *
        --------------------------------------------+
        BL 1 2 3 4 5 6 7 8 91011121314151617181920AB
                   CATEGORIES
```

Figure 7. Relative frequency histogram after 2048 iterations.

```
                    UNCERTAINTY MODEL
       --POLLUTANT CONCENTRATION   FOR A LEACHATE PLUME
       AFTER     2048ITERATIONS


   CONCENCENTRATION (MG/L)  MEAN (x) AND STANDARD DEVIATION (o)
     MEAN STD DEV           VERSES NUMBER OF ITERATIONS
.88E+01   .38E+01+....+....+....+....+....+....+....+....+....+....+....o..#.o
                 .    .    .    .    .    .    o  o o    .  o o  o o  o x    x
                 .    .    .    .    .    .  o .    .    o    .  x x  x .    .
                 .    .    .    .    .    .    .    .  o .    .    .    .    .
                 .    .    .    .    .  o o    .    .    .  x x    .    .    .
.87E+01   .37E+01+.......................................x...................+
                 .    .  o o  o o    o    .    .    .    .    .    .    .    .
                 .    .    .    .  o .    .    .    .    .    .    .    .    .
                 .    .    .    .    .    x    .    .  x .    .    .    .    .
                 .    .    .  x .    .  x .  x x    .    .    .    .    .    .
.85E+01   .35E+01+................................x.x........................+
                 .    .    x    x    x    .    .    .    .    .    .    .    .
                 .    .    .    .  x .    .    .    .    .    .    .    .    .
                 .    #    .    .    .    .    .    .    .    .    .    .    .
                 x    .  x .    .    .    .    .    .    .    .    .    .    .
.84E+01   .34E+01o..#.+....+....+....+....+....+....+....+....+....+....+....+
                81  243  405  567  729  891 1053 1215 1377 1539 1701 1863 2025
                                     ITERATIONS
```

Figure 8. Mean and standard deviation of chromium concentration distribution as a function of sample size (after 2048 iterations).

statistics can be accepted as representing estimates of the population statistics, and the simulation was terminated. The mean of this distribution is 8.78 mg/l compared to the deterministic modeling result of 8.38 mg/l.

For interpreting the results of a stochastic model in ground-water contamination studies, a cumulative frequency distribution is more useful (Figure 9). In this form, the ensemble of calculated concentrations and the corresponding probabilities can be interpreted as a risk analysis based on uncertainty in the model input parameters. In other words, there is a 50% probability that the predicted concentration is greater than or equal to 8.78 mg/l, which is the mean of the distribution. There is a 80% probability that the concentration is greater than or equal to 5.6 mg/l; a 5% probability that the concentration is greater than 14 mg/l.

Notice also that the possible range in concentration that could occur at the specified observation point lies between 0.09 and 22 mg/l. This range, of course, would change if any of the input values are modified. The most probable concentration is 8.78 mg/l as compared to 8.34 mg/l using a deterministic modeling approach.

SUMMARY

Monte Carlo simulations can provide a useful means of incorporating uncertainty in model input parameters into deterministic solute transport models. The approach used in this paper neglects uncertainties in the choice of solute transport equations, but does account for uncertainties in the values of model input parameters. This, as any other stochastic modeling approach, can lead to improved decision making if properly applied and interpreted. Unfortunately, stochastic modeling studies often lead to confusion and frustration since they produce probability distributions for the model output variables rather than a single deterministic value. The additional information may be difficult to interpret and also tends to require some judgement in the decision making process. Results are no longer black and white, but rather varying shades of grey.

In this brief paper we have tried to accomplish two objectives. The first is to point out that there are uncertainties in all ground-water flow and solute-transport modeling studies. These uncertainties may arise from, among other things, uncertainties in the model used to describe the physical system, and random fluctuations in loadings or in the unmeasured ambient conditions affecting the process. Uncertainties also may be caused by inadequate data collection techniques, including instrumentation and analysis, and uncertainties in the data used in the calculations. These stochastic features can be taken into account in deterministic models.

```
                 UNCERTAINTY MODEL
       --POLLUTANT CONCENTRATION   FOR A LEACHATE PLUME
       AFTER     2048ITERATIONS



         CUMULATIVE FREQUENCY HISTOGRAM FOR VALUES
PERCENT  GREATER THAN OR EQUAL TO LOWER CATEGORY VALUE
100.00+* * *
      I* * * *
      I* * * * * *
      I* * * * * * *
      I* * * * * * * *
 66.67+* * * * * * * *
      I* * * * * * * * *
      I* * * * * * * * * *
      I* * * * * * * * * *
      I* * * * * * * * * * *
 33.33+* * * * * * * * * * *
      I* * * * * * * * * * * *
      I* * * * * * * * * * * * *
      I* * * * * * * * * * * * * *
      I* * * * * * * * * * * * * * *
   .00+* * * * * * * * * * * * * * * * * * * * * *
       ------------------------------------------+
       BL 1 2 3 4 5 6 7 8 91011121314151617181920AB
                 CATEGORIES
```

Figure 9. Cumulative frequency distribution for chromium concentration after 2048 iterations.

Secondly, the results of stochastic modeling studies can be interpreted in more familiar terms such as risk analysis. Weather forecasts have been given in these terms for several years, and we all use them to make decisions as part of everyday life. With a little effort uncertainties in groundwater modeling studies can also be included in our decision making processes, either through the explicit use of stochastic models or through an awareness of the types and sources of uncertainty in deterministic models.

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AN EVALUATION OF THE PERFORMANCE OF ZONE OF SATURATION LANDFILLS IN WISCONSIN

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Abstract

A typical "zone-of-saturation" landfill in Wisconsin is developed by excavating into a saturated clay soil environment. The excavation functions as a discharge site by creating a depression in the water table, but does not fill with ground water because evaporation exceeds the rate of seepage from low-permeability clay deposits. Once the base and sidewalls of the excavation are covered with refuse ground water inflow and percolation from rainfall must be removed by the leachate collection system. The potential for leachate migration away from this type of landfill is minimized as long as inward hydraulic gradients are maintained at the base and perimeter of the facility.

Recent investigations have revealed that many zone-of-saturation sites are not situated in a "homogeneous" clay environment. Also, the unexpected buildup of leachate within several of the 12 landfills approved in Wisconsin using this design concept has led to the development of outward gradients. This paper provides a detailed performance evaluation of three of these sites where ground water contamination is occurring. Based on this evaluation, supplemented by data and observations from several of the other sites, recommendations are made for the investigation, design and operation of a zone-of-saturation landfill. Although there is insufficient data to fully evaluate their effectiveness, it is concluded that implementation of these recommendations will provide for an acceptable level of ground water protection at landfills using this design concept.

Introduction

In the late 1960's and early 1970's, as public concern for environmental protection mounted, regulatory agencies intensified their scrutiny of solid waste disposal sites. In Wisconsin, the State's Department of Natural Resources required site owners and operators to determine what effect their sites were having on surface water and ground water quality. The subsequent investigations documented that landfills situated in clay environments were developed in saturated soils, a fact which had not been recognized when these sites were initially constructed.

This discovery exposed a conflict between Wisconsin's existing landfill siting approach, which called for a minimum distance of 10 feet (3 m) between the base of a landfill and the water table, and actual practice in clay sites, where soils are typically saturated at or near the ground surface. Designers sought a way to retain the attenuative capacity of the clays and supply on-site cover material for wastes. As a result of these efforts, in 1975 the Department of Natural Resources developed guidelines which allowed landfills to be operated within the zone-of-saturation under controlled conditions (Glebs, 1980).

Guidelines call for a physical setting of homogeneous clay deposits with an in situ hydraulic conductivity of 1×10^{-6} cm/sec or less, maximum excavation of 30 feet (9.1 m), a minimum of 30 feet (9.1 m) of clay between the base of the fill and the water supply aquifer, a detailed hydrogeologic investigation, a leachate collection system and a leachate management plan.

The reasoning behind the 1975 guidelines can be summarized as follows: The hydraulic conductivity guideline of 1×10^{-6} cm/sec represents a value where ground water will seep into an excavation at a rate which is less than or equal to the rate of evaporation in Wisconsin. This ensures that when a saturated clay site is developed, major dewatering of the excavation will not be necessary. The depth of cut and the thickness of clay remaining beneath the base of the site are important for a number of reasons. First, while the guidelines call for "homogeneous" clay deposits, in practice this is seldom the case (Hendry, 1982; Prudic, 1982; Sterett and Edil, 1982; Fetter, 1980; Grisak et al., 1976; William and Farvolden, 1967). Geologic discontinuities (till joints and/or permeable seams of sand and silt) are commonly found in clayey glacial till and lacustrine deposits which can allow for rapid migration of contaminants away from a site in a saturated environment. The possibility of a direct connection to the water supply aquifer is reduced by specifying a relatively large thickness of clay beneath the base of the site. Second, limiting the depth of excavation minimizes uplift pressures on the base and sidewalls. Third, because the clay is saturated its attenuative capacity is greatly reduced over unsaturated soils. This requires an added thickness of clay. A detailed hydrogeologic investigation should define the variability existing within the clay deposits. An efficient leachate collection system and a plan for the ultimate disposal of the collected liquid are critical to ensure that leachate head buildup is minimized because zone-of-saturation sites will begin producing leachate almost immediately.

Since 1975, the Department of Natural Resources has approved 12 landfills utilizing the zone-of-saturation design concept. Figure 1 shows the general location of these sites. In general, all 12 of the sites are located in saturated clay environments comprised of glacial

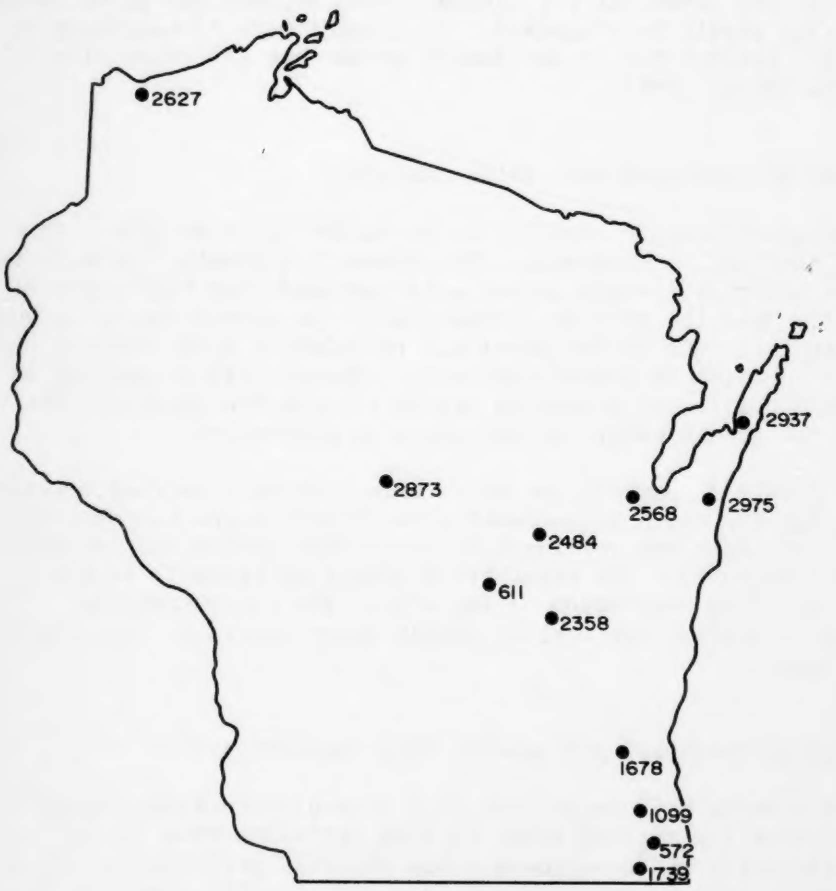
2627
2937
2873
2568
2975
2484
611
2358
1678
1099
572
1739

FIGURE 1: LOCATION OF THE 12 MAJOR ZONE-OF-SATURATION LANDFILLS

till and associated lacustrine deposits over fractured sedimentary or crystalline rock.

This paper documents problems concerning the unexpected buildup of leachate and ground water contamination which have developed at a number of these sites. Recommendations are made based on either a qualitative evaluation or actual field data for the investigation, design and operation of a zone-of-saturation landfill. If these recommendations are implemented, the potential for leachate head buildup and ground water contamination should be minimized. Additional work is necessary to evaluate the sensitivity of the design parameters affecting site performance (Quinn, 1983).

Development of A Zone-of-Saturation Landfill

A zone-of-saturation landfill is developed by excavating into a saturated clay soil environment. The excavation remains dry because the rate of evaporation exceeds ground water seepage from tight clay deposits (Figure 2). Once the base and sidewalls of the excavation are covered with refuse inflowing ground water and percolation from rainfall have to be removed to maintain inward gradients. Theoretically, as long as both vertical and horizontal gradients remain towards the landfill, the potential for ground water contamination is eliminated.

Water levels in observation wells were used to construct a water table map for the Omega Hills South site (#1648) where the excavation had already taken place but the leachate collection system had not yet been installed (Figure 3). The water table sloped principally to the southeast prior to excavation of the site. The excavation has significantly altered the shallow ground water system so that now flow is radially inward.

Typical Design Variables and General Site Characteristics

Figure 4 shows a cross section of a zone-of-saturation landfill. The data represents the typical range of site variables from the 12 zone-of-saturation sites approved using the 1975 guidelines. The most significant variation occurs in the in situ hydraulic conductivity of the clayey glacial till and lacustrine deposits.

The results of laboratory and field tests used to characterize the glacial sediments at these sites are presented in Table 1. This table shows that actual hydraulic conductivities of the glacial deposits are 1 to 3 orders of magnitude greater than values obtained from laboratory tests on either undisturbed or recompacted samples. Stratified drift (outwash and/or ice contact deposits of well-sorted sand and gravel) seams of sand and silt and/or secondary porosity from joints in the saturated clayey glacial till and lacustrine deposits account for these differences. Table 2 summarizes additional information on each site.

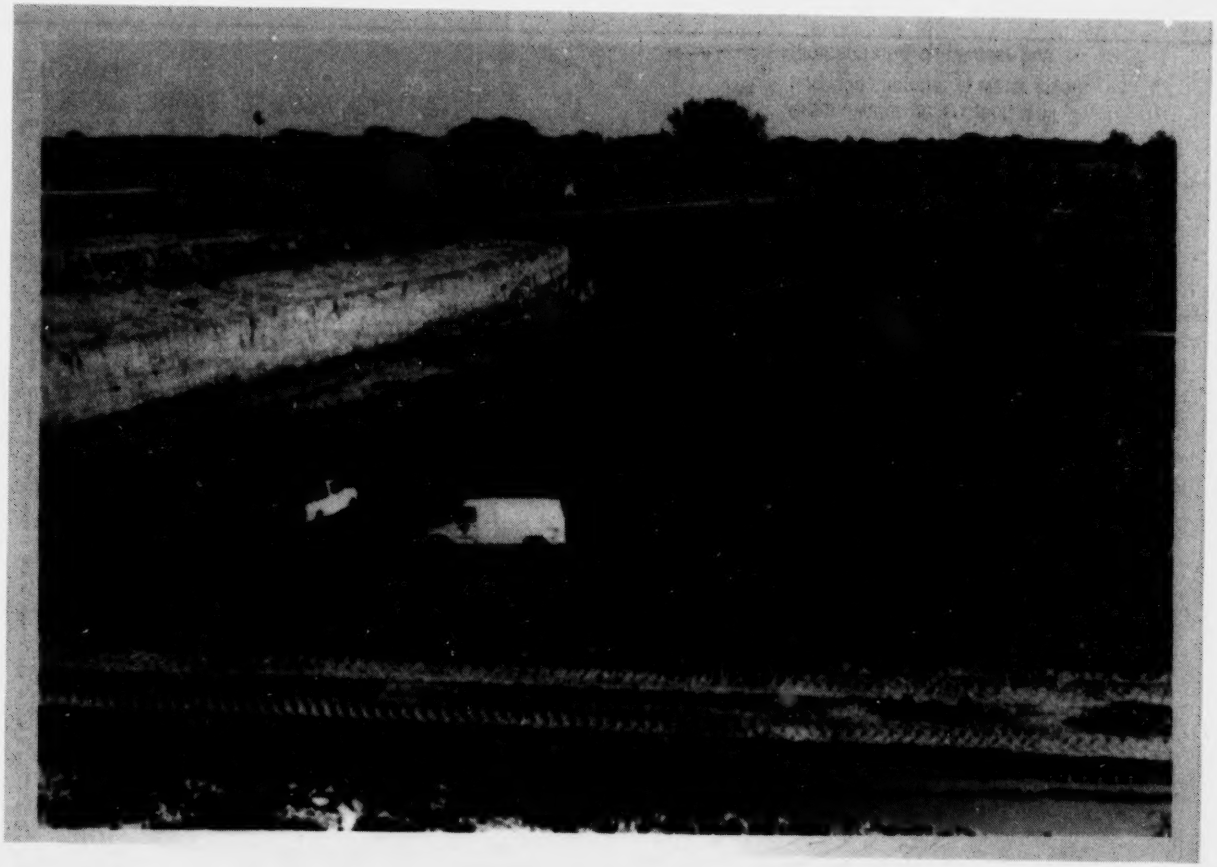

FIGURE 2: A COMPLETED EXCAVATION OF A CLAY SITE WHICH EXTENDS APPROXIMATELY 20 FEET BELOW THE WATER TABLE

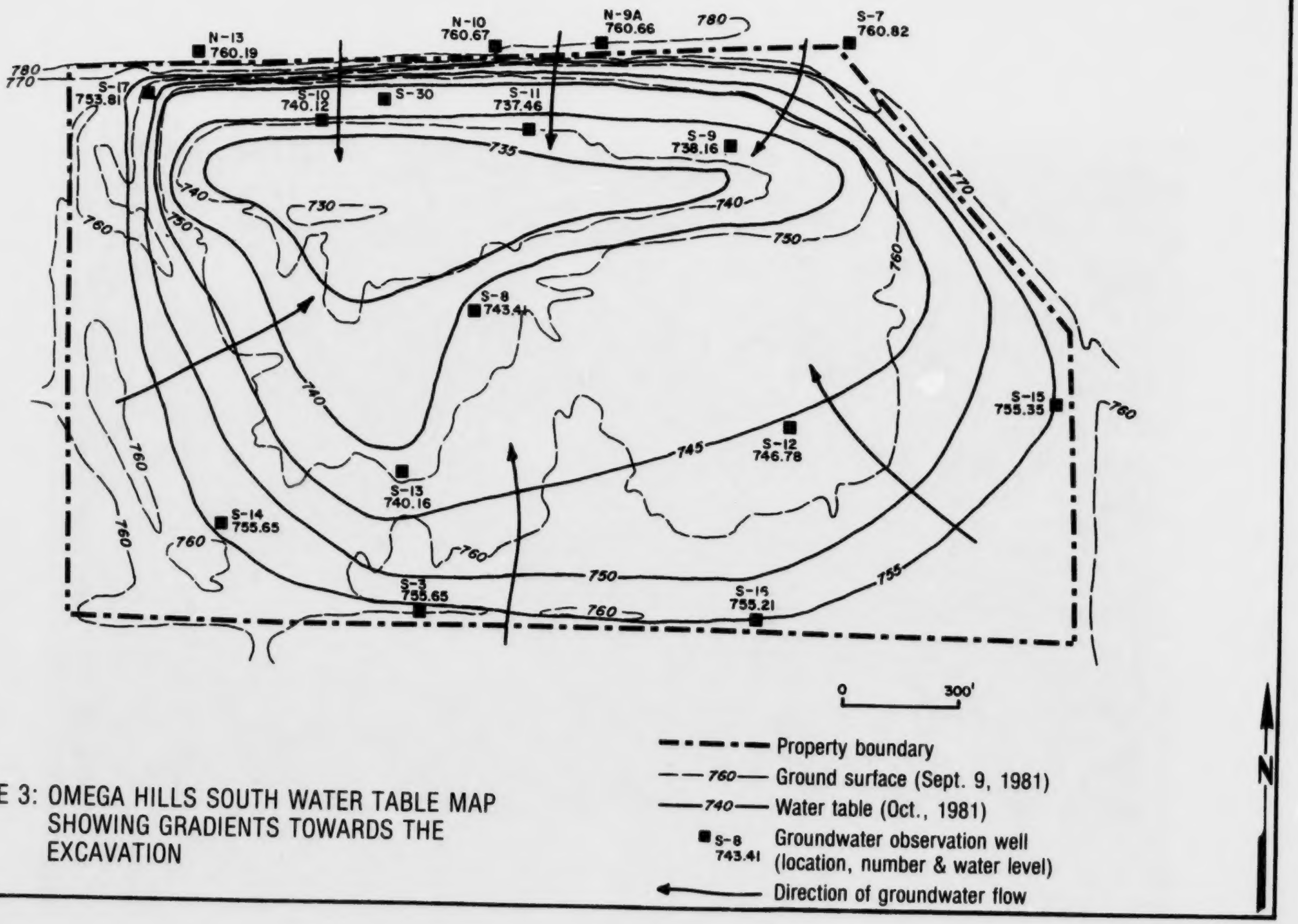
N-13
760.19
N-10
760.67
N-9A
760.66
780
S-7
760.82
780
770
S-17
753.81
S-10
740.12
S-30
S-11
737.46
S-9
738.16
735
740
730
770
740
750
760
750
760
S-8
743.41
740
S-15
755.35
760
745
S-12
746.78
S-13
740.16
S-14
755.65
760
760
760
760
750
755
S-3
755.65
S-16
755.21
760
0
300'
Property boundary
760 Ground surface (Sept. 9, 1981)
740 Water table (Oct., 1981)
S-8
743.41
Groundwater observation well (location, number & water level)
Direction of groundwater flow
N

FIGURE 3: OMEGA HILLS SOUTH WATER TABLE MAP SHOWING GRADIENTS TOWARDS THE EXCAVATION

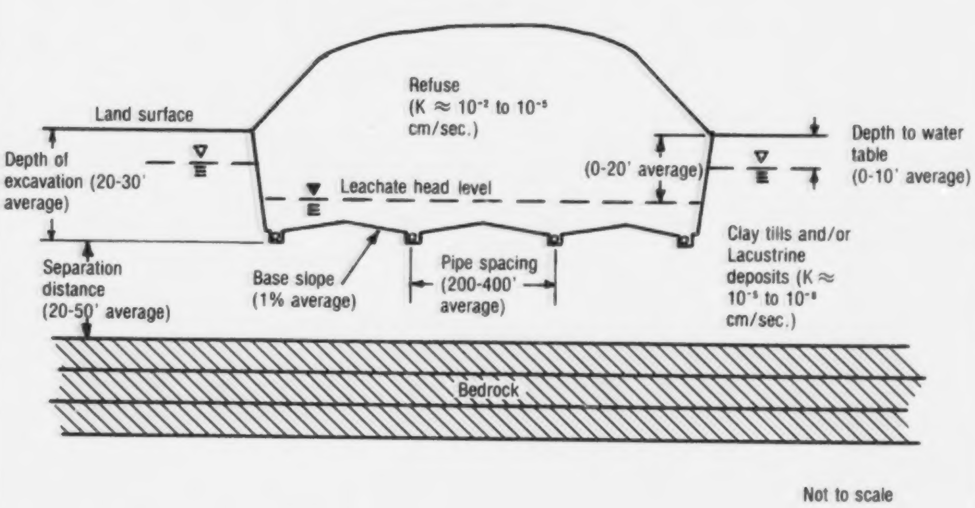
Refuse
(K ≈ 10^{-2} to 10^{-5} cm/sec.)
Land surface
Depth to water table (0-10' average)
Depth of excavation (20-30' average)
(0-20' average)
Leachate head level
Clay tills and/or Lacustrine deposits (K ≈ 10^{-5} to 10^{-8} cm/sec.)
Separation distance (20-50' average)
Base slope (1% average)
Pipe spacing (200-400' average)
Bedrock
Not to scale

FIGURE 4: CROSS-SECTION OF A ZONE-OF-SATURATION LANDFILL AND THE TYPICAL RANGE OF SITE VARIABLES

TABLE 1: SOIL CHARACTERISTICS OF 12 ZONE-OF-SATURATION LANDFILLS LOCATED IN WISCONSIN

License		P200 Content (%)			Clay Size (.005mm) (%)			Liquid Limits (LL)			Plastic Index (PI)			Field Permeability (cm/sec)			Laboratory Permeability (cm/sec)		
#	Name	No. of Values	Range	Avg.	No. of Values	Range	Avg.	No. of Values	Range	Avg.	No. of Values	Range	Avg.	No. of Values	Range	Median	No. of Values	Range	Median
2568	Brown County West	70	39-100	79	--	--	--	81	0-54	28	81	0-30	10	30	$6x10^{-8}$ to $6x10^{-4}$	--	38	$3x10^{-8}$ to $8x10^{-5}$	$3x10^{-7}$
2937	Door County	8	38-70	62	8	32-42	38	2	34-39	37	2	8-15	12	9	$2x10^{-7}$ to $2x10^{-6}$	$5x10^{-7}$	8	$7x10^{-9}$ to $1x10^{-7}$	$1x10^{-8}$
2358	Fond du Lac County	67	3-100	72	39	0-78	41	47	0-56	32	47	0-35	15	--	--	--	9	$4x10^{-9}$ to $8x10^{-8}$	$7x10^{-8}$
2975	Kewaunee County	46	2-75	62	7	6-36	21	7	22-30	26	7	10-16	14	8	$5x10^{-8}$ to $6x10^{-6}$	$6x10^{-7}$	35	$4x10^{-9}$ to $9x10^{-6}$	$2x10^{-8}$
572	Land Reclamation	46	8-100	75	46	0-73	30	46	0-44	24	46	0-26	10	11	$1x10^{-7}$ to $3x10^{-3}$	$3x10^{-6}$	22	$5x10^{9}$ to $7x10^{-4}$	$3x10^{-8}$
1099	WMI-Metro	48	74-100	85	40	42-75	51	49	17-48	33	49	3-27	15	15	$6x10^{-9}$ to $6x10^{-4}$	$3x10^{-7}$	2	$2x10^{-8}$	$2x10^{-8}$
1678	WMI-Omega Hills	188	1-100	66	85	5-65	38	63	14-44	28	63	2-25	13	16	$2x10^{-6}$ to $2x10^{-3}$	$7.5x10^{-6}$	27	$3x10^{-9}$ to $5x10^{-3}$	$9x10^{-8}$
2484	Outagamie County	59	26-99	83	59	3-88	50	51	15-67	36	51	4-46	19	39	$2x10^{-7}$ to $5x10^{-4}$	$1x10^{-5}$	36	$4x10^{-9}$ to $7x10^{-7}$	$3x10^{-8}$
1739	WMI-Pheasant Run	38	72-99	87	38	10-85	40	23	20-45	28	23	4-22	10	13	$1x10^{-7}$ to $2x10^{-5}$	$6x10^{-7}$	5	$3x10^{-8}$ $1x10^{-6}$	$6x10^{-8}$
2627	City of Superior	3	95-97	96	3	64-84	76	4	64-82	74	4	41-59	50	10	$3x10^{-6}$ to $2x10^{-5}$	$3x10^{-6}$	4	$1x10^{-8}$ to $8x10^{-8}$	$1x10^{-8}$
2873	Weyerhaeuser Company	6	4-98	61	9	5-52	33	7	19-47	35	7	6-26	19	2	$2x10^{-6}$ to $3x10^{-6}$	$3x10^{-6}$	8	$2x10^{-8}$ to $5x10^{-6}$	$1x10^{-7}$
611	Winnebago County	370	3-97	49	--	—	—	-4	38-39	38	4	20-22	--	52	$9x10^{-8}$ to $2x10^{-3}$	$2x10^{-6}$	35	$3x10^{-9}$ to $2x10^{-4}$	$2x10^{-7}$

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TABLE 2: GENERAL SITE CHARACTERISTICS OF 12 ZONE-OF-SATURATION LANDFILLS LOCATED IN WISCONSIN

License No.	Name	Total Design Volume Cubic Yards (CY)	Principle Waste Types Received(1)	Leachate Collection System Partial	Leachate Collection System Full	Date Filling Began	Site Size (Acres)	Design Volume Filled (%)
2568	Brown County West	4×10^6	MSW, IND		X	1977	50	30
2937	Door County	7.4×10^5	MSW, IND		X	1982	17.4	20
2358	Fond du Lac County	5×10^5	MSW, IND		X	1978	16	90
2975	Kewaunee County	5.2×10^5	MSW, IND		X	1983	15	3
572	Land Reclamation	9.5×10^6	MSW, IND, HW	X		pre-1970	82	50
1099	WMI-Metro	9×10^6	MSW, IND, HW	X		pre-1970	96	25
1678	WMI-Omega Hills	15×10^6	MSW, IND, HW	X		1971	166	60
2484	Outagamie County	3.2×10^6	MSW, IND		X	1975	47	75
1739	WMI-Pheasant Run	1.6×10^6	MSW, IND	X		pre-1967	35	25
2627	City of Superior	6.0×10^5	MSW, IND		X	1976	19.6	70
2873	Weyerhaeuser Company	1×10^5	IND		X	1978	7.5	100
611	Winnebago County	5.5×10^6	MSW, IND	X		pre-1970	94	80

(1) MSW = Municipal Solid Waste; IND = Non-Hazardous Industrial Waste; HW = Hazardous Waste

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Documentation of Problems

Recent hydrogeologic investigations have documented that many zone-of-saturation sites are not situated in a "homogeneous" clay environment. Also, the unexpected buildup of leachate within several of the 12 landfill sites approved since 1975 using this design concept has led to the development of outward gradients and severe ground water contamination. Three of these sites -- Omega Hills North, Outagamie County and Winnebago County have been selected to illustrate these problems and are discussed in detail below. In all three cases the site owners have implemented remedial actions to help alleviate the environmental impacts. Studies are in progress to define the existing site conditions in more detail and to develop cleanup programs which will include design and/or operational changes.

Omega Hills North

The Omega Hills North landfill (#1678) provides an excellent example of the problems which can develop at a zone-of-saturation landfill. The site has been licensed since 1971. In early 1978, the Department of Natural Resources completed an Environmental Impact Statement (EIS) which concluded that the 83 acre (33.6 ha) site was suitable for an additional 6 million cubic yards (4.6 million m^3) of municipal and industrial wastes, some of which were classified as hazardous.

The site is situated in glacial till deposits primarily consisting of silt and clay-sized materials. Site investigations revealed some lenses and more extensive glaciofluvial deposits of sand and gravel. The unconsolidated glacial sediments are typically saturated to within 10 feet (3 m) of the land surface. Shallow ground water flow generally follows the topography and was in a southeasterly direction prior to site development.

The uppermost bedrock unit beneath the site consists of Niagaran dolomite of Silurian age. The dolomite is moderately fractured and up to 200 feet thick. Private wells in the vicinity of the site tap the bedrock aquifer for their water supply.

The approval for the Omega Hills landfill required that permeable deposits encountered during site construction or base soil certification be excavated a minimum of 5 feet (1.5 m) and be backfilled with compacted clay. In many cases permeable deposits were identified by ground water discharge, their large areal extent or changes in soil color. In one area of the site sandy deposits were not properly sealed with clay and leachate escaped into the bedrock aquifer. Figure 5 is a generalized cross section of this area following implementation of temporary remedial actions in August, 1982.

The cross section shows that the erosional surface of the bedrock is much closer to the base of the site than originally thought in 1978. The surface of the dolomite drops off sharply to the east, west and south which may explain why it was not discovered during earlier site investigations.

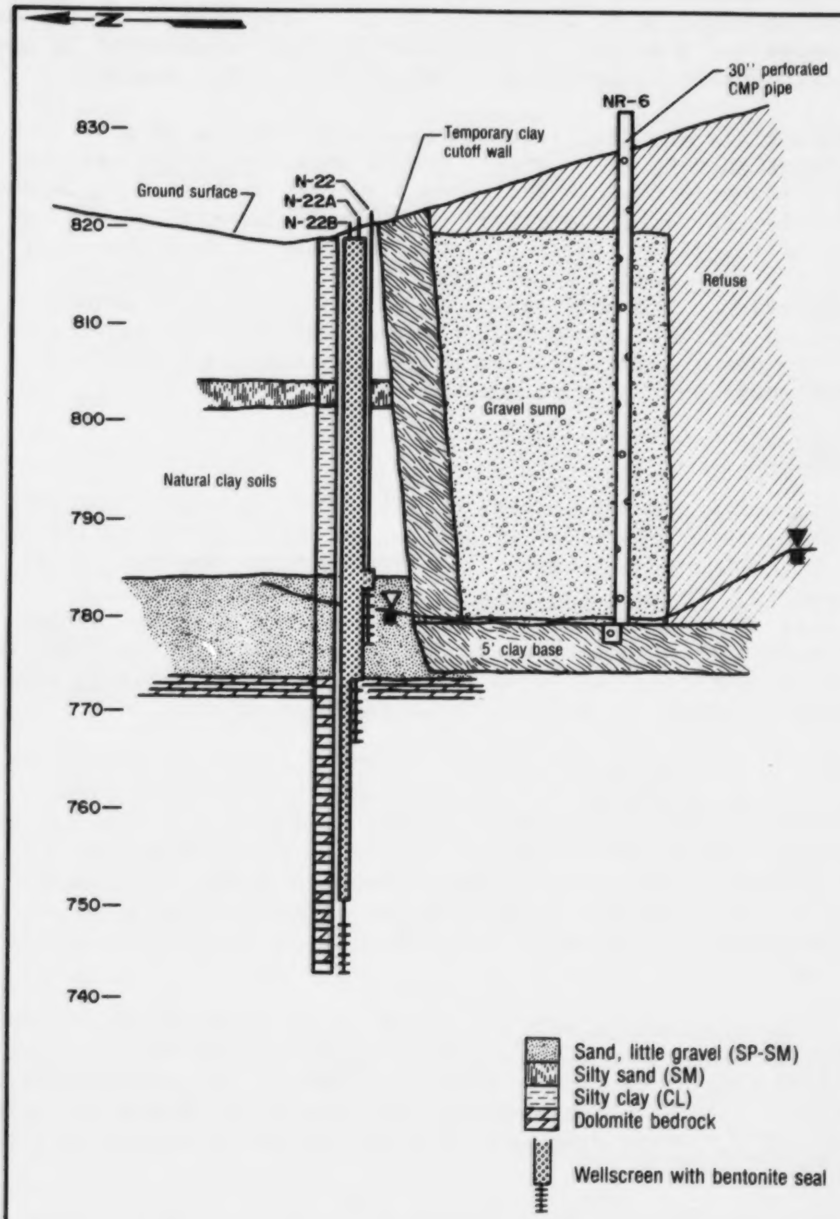
N
30'' perforated CMP pipe
NR-6
830
Temporary clay cutoff wall
N-22
Ground surface
N-22A
820
N-22B
Refuse
810
Gravel sump
800
Natural clay soils
790
780
5' clay base
770
760
750
740
Sand, little gravel (SP-SM)
Silty sand (SM)
Silty clay (CL)
Dolomite bedrock
Wellscreen with bentonite seal

FIGURE 5: GENERAL CROSS-SECTION THROUGH WELL NEST N-22 AND RISER NR-6 SHOWING GEOLOGIC CONDITIONS AND TEMPORARY REMEDIAL WORK

Water quality results from well nest N-22 shown in Figure 5 and an unaffected upgradient well (N-3) document that ground water has been severely contaminated in this area (Table 3). Placement of a clay cutoff wall and removal of leachate have resulted in some improvement in ground water quality, however, significant contamination still remains.

The Omega Hills landfill also represents an example of a zone-of-saturation site where inward ground water gradients have not been maintained. Figure 6 depicts the water table as it existed in October, 1982. Water levels in leachate head wells installed through the refuse to the base of the fill (Figure 7) were also used to draw the water table map because the base grades for a majority of the site are within the zone-of-saturation. The acceptance of large quantities of liquid waste and the lack of leachate management alternatives were the major factors leading to the development of an excessive leachate head within the landfill and subsequent outward gradients.

Outagamie County

An example of a zone-of-saturation site where joints have rendered fairly uniform clayey glacial till and lacustrine deposits relatively permeable is the Outagamie County landfill (#2484). The Department of Natural Resources licensed the site in 1975. Initially, 26 acres (10.5 ha) were used for disposal of nonshredded and shredded solid wastes and large quantities of papermill sludge. In late 1981, the landfill was expanded to approximately 47 acres (19 ha) and the total design capacity was increased to about 3.2 million cubic yards (2.45 million m^3).

The site is located in an area of relatively flat and gently sloping topography with no prominent relief features. Two clayey glacial till units dominate the subsurface stratigraphy. A relatively thin layer of varved glaciolacustrine silty clay with thin to medium seams of peat and pieces of partially decayed wood (Twocreekan Age forest bed deposit) separates the 2 till units. Underlying the unconsolidated glacial sediments at a depth of 55 to 70 feet (16.8 to 23.3 m) are dolomites of Ordivician age.

Water table observation wells indicate that saturated conditions exist within 10 feet (3 m) of the land surface. Ground water within the unconsolidated glacial deposits primarily flows to the south-southeast. Vertical ground water gradients within the clay soils generally range between 0.6 and 0.9 ft/ft downward, although values in excess of 1 have been recorded.

The median *in situ* hydraulic conductivity of the clayey glacial deposits is 3 orders of magnitude greater than that predicted by laboratory permeability tests. Test pits excavated in the proposed 21 acre (8.5 ha) expansion area in March, 1981 revealed joints in the glacial deposits. Based on this evidence and the results of several in-field permeability tests, the approval for the landfill expansion required overexcavation and recompaction of the base and sidewalls of the site to provide a uniform clay barrier with a maximum permeability of 1×10^{-7} cm/sec between the waste and the surrounding saturated environment. Excavation of the expansion area exposed vertical sets of

TABLE 3: WATER QUALITY IN WELL NEST N-22 AND LEACHATE QUALITY IN RISER NR-6

	Sampling Dates							
	12/14/81	1/18/82	6/15/82	7/13/82	7/28/82	9/82	11/10/82	12/82
WELL N-22								
pH	6.2	6.3		6.34	6.47		6.4	
Conductivity	8,480	6,999		7,050	6,000		3,200	
Chloride	1,240	872		786	276		61	
COD	8,910	10,800		8,850	9,230		2,270	
Total Alkalinity	6,520	2,340		2,526	3,366		447	
Total Hardness	6,640	5,680		3,167	4,710		1,600	
Sulfate		5		5	13.7		5	
TOC	4,250	2,600		3,142	3,045		1,440	
Dissolved Iron	101.5			122	20.6		42.1	
Sodium	593	398		240	190		120	
Manganese	3.27	3.23		142	1.06		1.12	
WELL N-22A								
pH				6.42	6.50		6.4	
Conductivity				1,580	1,450		1,600	
Chloride				25	193		16	
COD				43.9	21		47	
Total Alkalinity				850	1,050		834	
Total Hardness				790	990		896	
Sulfate				60	5.0		48	
TOC				24	18		2.3	
Dissolved Iron				24	20.6		16.8	
Sodium				3.33	3.14		6.0	
Manganese				0.90	0.64		0.66	
WELL N-22B								
pH				6.48	6.63		6.4	
Conductivity				1,360	1,130		1,330	
Chloride				10	5		7	
COD				14.6	10		29	
Total Alkalinity				775	745		721	
Total Hardness				711	778		834	
Sulfate				54	53		75	
TOC				124	5.1		2.4	
Dissolved Iron				8.34	4.27		4.44	
Sodium				4.92	0.5		5.0	
Manganese				0.66	0.64		0.85	
RISER NR-6								
pH						6.55		6.55
Conductivity						20,000		19,000
COD						29,000		3,140
Total Alkalinity						10,800		—
Total Hardness						5,893		4,770
Total Iron						402		308
BOD						12,900		25,800

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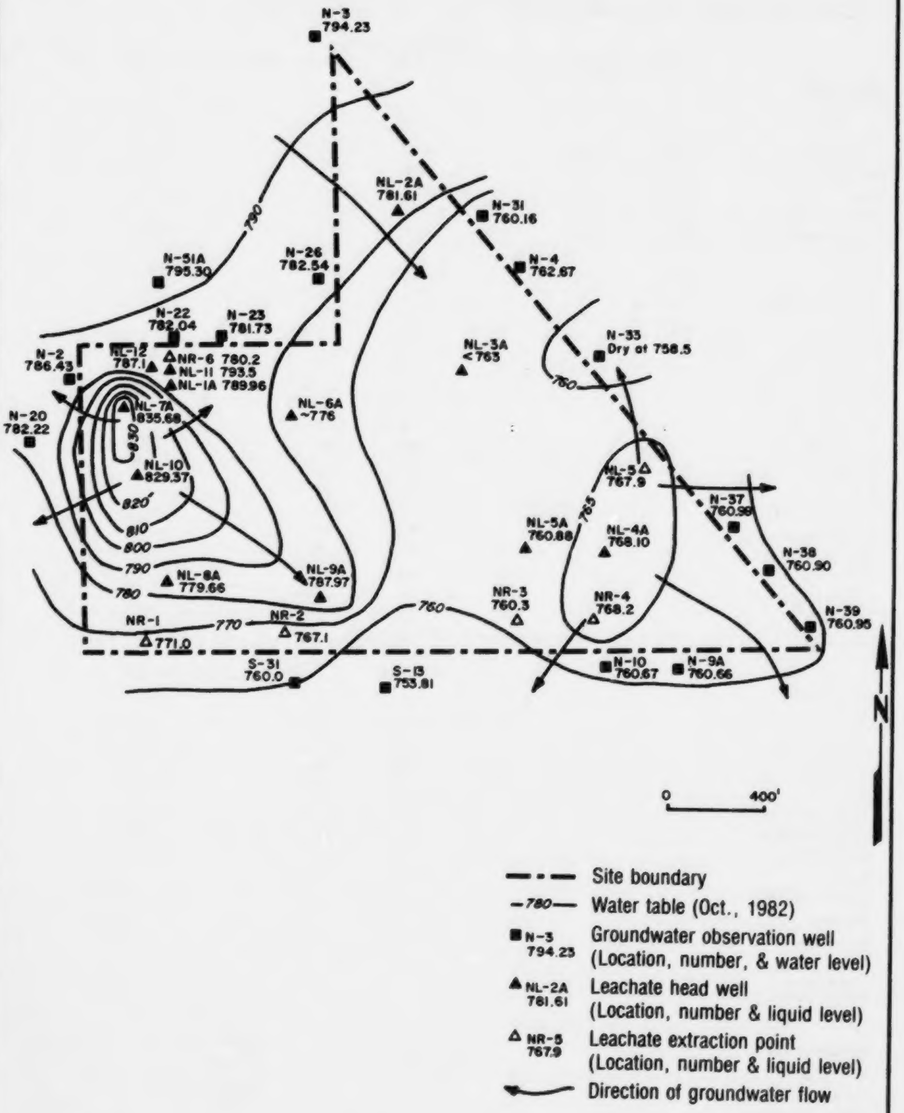

N-3
794.23
NL-2A
781.61
N-31
760.16
N-4
762.67
N-51A
795.30
790
N-26
782.54
N-22
782.04
N-23
781.73
N-2
786.43
NL-12
787.1
NR-6 780.2
NL-11 793.5
NL-1A 789.96
NL-7A
835.68
830
N-20
782.22
NL-10
829.37
820
810
800
790
780
770
760
765
NL-6A
~776
NL-8A
779.66
NL-9A
787.97
NR-1
771.0
NR-2
767.1
S-31
760.0
S-13
753.81
NL-3A
<763
N-33
Dry at 758.5
NR-5
767.9
N-37
760.98
NL-5A
760.88
NL-4A
768.10
N-38
760.90
NR-3
760.3
NR-4
768.2
N-39
760.95
N-10
760.67
N-9A
760.66
N
0
400'
Site boundary
780 Water table (Oct., 1982)
N-3 794.23 Groundwater observation well (Location, number, & water level)
NL-2A 781.61 Leachate head well (Location, number & liquid level)
NR-5 767.9 Leachate extraction point (Location, number & liquid level)
Direction of groundwater flow

FIGURE 6: OMEGA HILLS NORTH WATER TABLE MAP SHOWING OUTWARD GRADIENTS

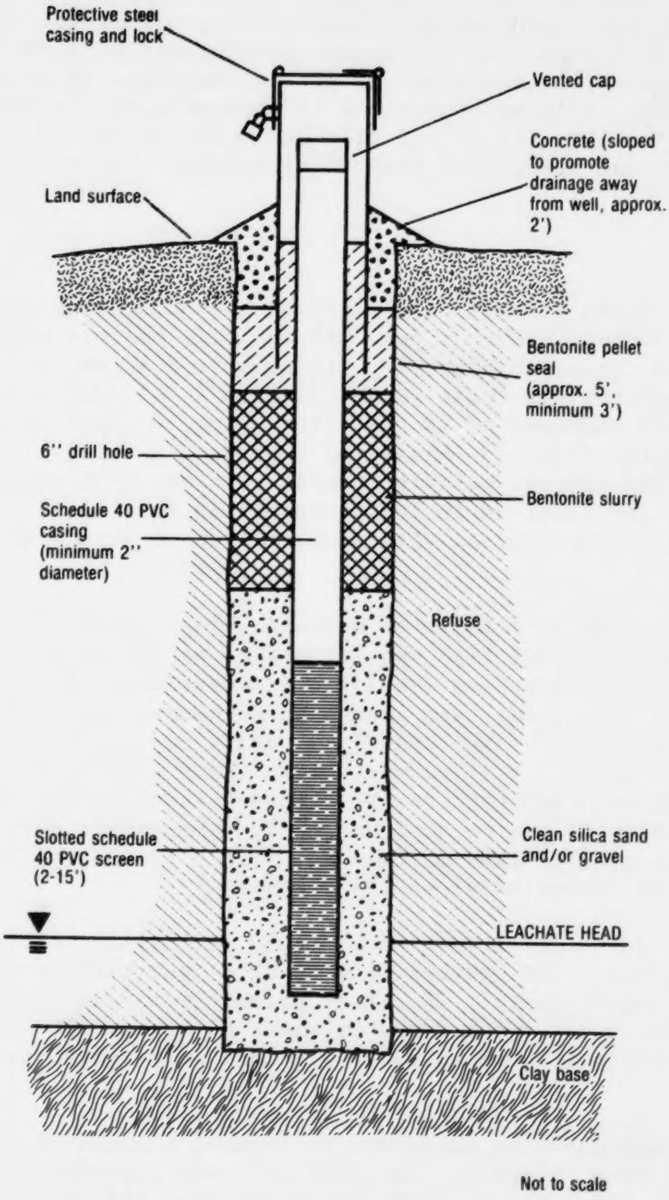

Protective steel casing and lock
Vented cap
Concrete (sloped to promote drainage away from well, approx. 2')
Land surface
Bentonite pellet seal (approx. 5', minimum 3')
6'' drill hole
Bentonite slurry
Schedule 40 PVC casing (minimum 2'' diameter)
Refuse
Slotted schedule 40 PVC screen (2-15')
Clean silica sand and/or gravel
LEACHATE HEAD
Clay base
Not to scale

FIGURE 7; TYPICAL CONSTRUCTION DETAILS OF A LEACHATE HEAD MONITORING WELL orthogonal joints, with apertures ranging from 0.06 to 0.375 inches (0.15 to 0.95 cm) wide extending down through the interglacial layer of peat (Figure 8) to a depth of at least 20 feet (6 m).

Available data show that gas pressures as high as 15 pounds per square inch (267.8 Kg/m^2) and leachate head buildup within the initial 26 acre (10.5 ha) fill area have led to the development of outward gradients and ground water contamination in the immediate vicinity of the site. The acceptance of large quantities of papermill sludge with an average solids content of 20%, use of clay soils for daily and intermediate cover, and inadequate removal of leachate led to these problems.

Winnebago County

The Winnebago County landfill (#611) is also experiencing problems due to excessive leachate head buildup. At the time of this writing, 71 acres (28.7 ha) are filled to capacity with another 8 acres (3.2 ha) active. Approximately 3,000 cubic yards (2,300 m^3) of municipal and industrial waste are disposed of at the site daily. Of that total, almost 960 cubic yards (730 m^3) consist of sludges generated by paper industries within the county. (Environmental problems at several private papermill sludge sites resulted in their closure in the late 70's. This led to co-disposal of the sludges with municipal waste at the County facility).

Flat topography, characteristic of a glacial lake plain, dominates the local landscape. Glacial till and lacustrine sediments of reddish-brown to brown silty clay overlie overconsolidated stratified and unstratified drift. The silty clay ranges in thickness from 5 feet to over 20 feet (1.5 to 6.1 m). The underlying drift is very heterogeneous in nature and consists of a grey to grey-brown sandy silt to silty sandy clay till. Excavations completed in 1983 exposed an extensive buried beach ridge consisting of coarse sand capped by wave-washed gravel. This extremely permeable glacial lake deposit was missed by the initial site investigations which consisted of 46 soil borings.

Ordivician age dolomite is located 40 to 50 feet (12.2 to 15.2 m) below the land surface. The bedrock surface dips generally to the south-southeast at approximately 2%. Most private wells in the area utilize the dolomite aquifer for domestic water supplies. The water table is present within 10 feet (3 m) of the land surface and slopes to the northwest.

The site has been used for solid waste disposal since 1971 when 25 acres (10.1 ha) were licensed. The fill area was expanded by an additional 30 acres (12.1 ha) in 1977 and by 40 acres (16.2 ha) in 1981. The first area to be designed with base grades below the water table (Site B) is divided into 4 distinct cells. The approval for Site B required a minimum base slope of 1%, a maximum leachate flow distance of 150 feet (45.7 m) and leachate removal at a rate sufficient to maintain a "dry base" condition. The first 2 cells (B-1 and B-2) were used for disposal of primarily municipal solid waste. Head wells installed in these 2 cells show leachate levels from 0 to 12 feet (0 to 3.7 m) above

FIGURE 8: FRACTURED CLAY DEPOSITS AT THE OUTAGAMIE COUNTY LANDFILL SITE

the base of the cells (Table 4). In Cells B-3 and B-4, levels in leachate head wells exceed 25 feet (7.6 m) in places. Well LMB-4A is installed directly over a leachate collection line which probably accounts for the minimal head buildup of less than 2 feet (0.61 m).

The development of leachate head buildup in Site B (Figure 9) led to a concern for ground water quality impacts. Figures 10 and 11 are plots of the ground water quality in well nest P-5, located approximately 25 feet (7.6 m) northwest of Cell B-4. Wells P-5-20 and P-5-40 are both screened within the silty sandy clay till at 19.5 and 29.5 feet (5.9 and 9 m) below the land surface. Well P-5-60 is screened within the dolomite bedrock approximately 48 feet below the ground surface. It is readily apparent from the graphs that the ground water in the vicinity of these wells is contaminated. Ground water degradation in the area of the P-5 well nest began about 1 year after disposal operations began in Cell B-4. This contamination led to additional investigations of head levels which confirmed the excessive buildup of leachate in Site B. Although the degradation of ground water in well nest P-5 occurred at about the same time as the leachate head buildup began in Site B, the contamination may be coming from either Cell B-4, Site A-1 or both. The high leachate heads within the site are attributed to the disposal of large volumes of papermill sludge. Not only did these wastes add additional liquids to the site, but the sludge has likely reduced the overall permeability of the waste mass, which in turn would reduce the efficiency of the leachate collection system.

Discussion

This paper documents 2 major problems currently exist at several zone-of-saturation landfills in Wisconsin. First, on-site investigations at many of these sites have shown that they are not situated in a "homogeneous" clay environment. Second, inward hydraulic gradients, which are basic to the design of zone-of-saturation landfills, do not exist in a number of cases. These factors have led to ground water contamination at a number of facilities and to concern about potential ground water deterioration at several others.

Rinaldo-Lee and Nichols (1979) documented that ground water impacts from a zone-of-saturation landfill situated in uniform glacial clay sediments was confined within 100 feet (30.5 m) along a flow line from the refuse. However, in the previous section we presented information which confirms that leachate will migrate preferentially through permeable discontinuities present in clayey glacial till and lacustrine deposits. For this reason, it is essential that the heterogeneities in a saturated clay soil environment be defined as accurately as possible prior to site development. If site investigations document that extensive permeable deposits such as sand and silt are not present, the site may be suitable for development under the zone-of-saturation design concept.

While a detailed hydrogeologic investigation should define any major permeable deposits, it will not detect all of the minor discontinuities. Therefore, the best method for ensuring that a uniform clay barrier

TABLE 4: LEACHATE HEAD LEVELS THROUGH TIME IN SITE B AT THE WINNEBAGO COUNTY LANDFILL(1)

WELL	1980				1981						1982									
NO.	3/12	6/11	9/26	12/17	3/12	6/16	6/18	9/14	9/15	12/15	4/14	6/24	6/25	7/1	8/12	9/13	9/22	10/13	11/24	12/13
LM-B1A	Dry	Dry	Dry	Dry	Dry	Dry			Dry	Dry		Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry
LM-B2A	Dry	770.89	770.9	Dry	Dry		772.5		772.5	771.0	770.5		Gas	Gas	Gas	Gas		Gas	Gas	
LM-B2C												771.03	770.73	770.4		772.23		772.53	772.33	
LM-B3A						781.26		783.7		784.0	784.75		784.75	784.77	785.01	785.5	785.55	785.8	786.1	785.1
LM-B3B												778.98		766.55	Gas	Gas		Gas	Gas	
LM-B4A												761.32		761.14	761.29	761.52		761.52	761.32	
LM-B4B														Gas	Gas	Gas	Gas		Gas	Gas
LM-B4C1												785.0	787.39	788.47	790.14	791.0		791.0	790.9	
LM-B4C2												790.29		790.51	791.82	792.59		793.79	793.99	
LM-A1																			Dry	

(1) Base grades in Site B range from 762 to 766.

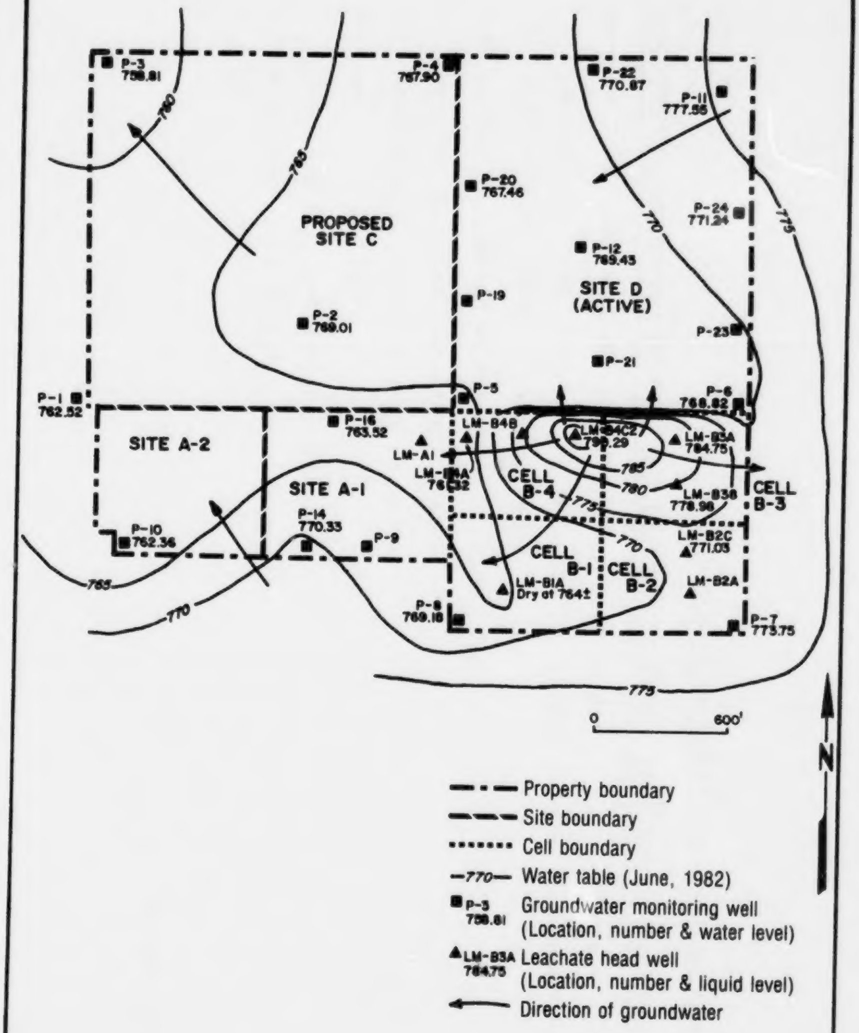

P-3
758.81
760
P-4
767.90
P-22
770.87
P-11
777.56
765
P-20
767.46
PROPOSED
SITE C
770
775
P-24
771.24
P-12
769.43
P-19
SITE D
(ACTIVE)
P-2
769.01
P-23
P-21
P-1
762.52
P-5
P-6
768.82
P-16
763.52
LM-B4B
LM-B4C2
790.29
LM-B3A
784.75
SITE A-2
LM-A1
LM-B4A
76_.32
785
CELL
B-4
780
SITE A-1
775
LM-B3B
778.96
CELL
B-3
P-14
770.33
P-10
762.36
P-9
770
LM-B2C
771.03
CELL
B-1
LM-B1A
Dry at 764±
CELL
B-2
LM-B2A
765
770
P-8
769.18
P-7
773.75
775
0
600'
N
Property boundary
Site boundary
Cell boundary
770
Water table (June, 1982)
P-3
758.81
Groundwater monitoring well
(Location, number & water level)
LM-B3A
784.75
Leachate head well
(Location, number & liquid level)
Direction of groundwater

FIGURE 9: WINNEBAGO COUNTY WATER TABLE MAP
SHOWING OUTWARD GRADIENTS IN SITE B

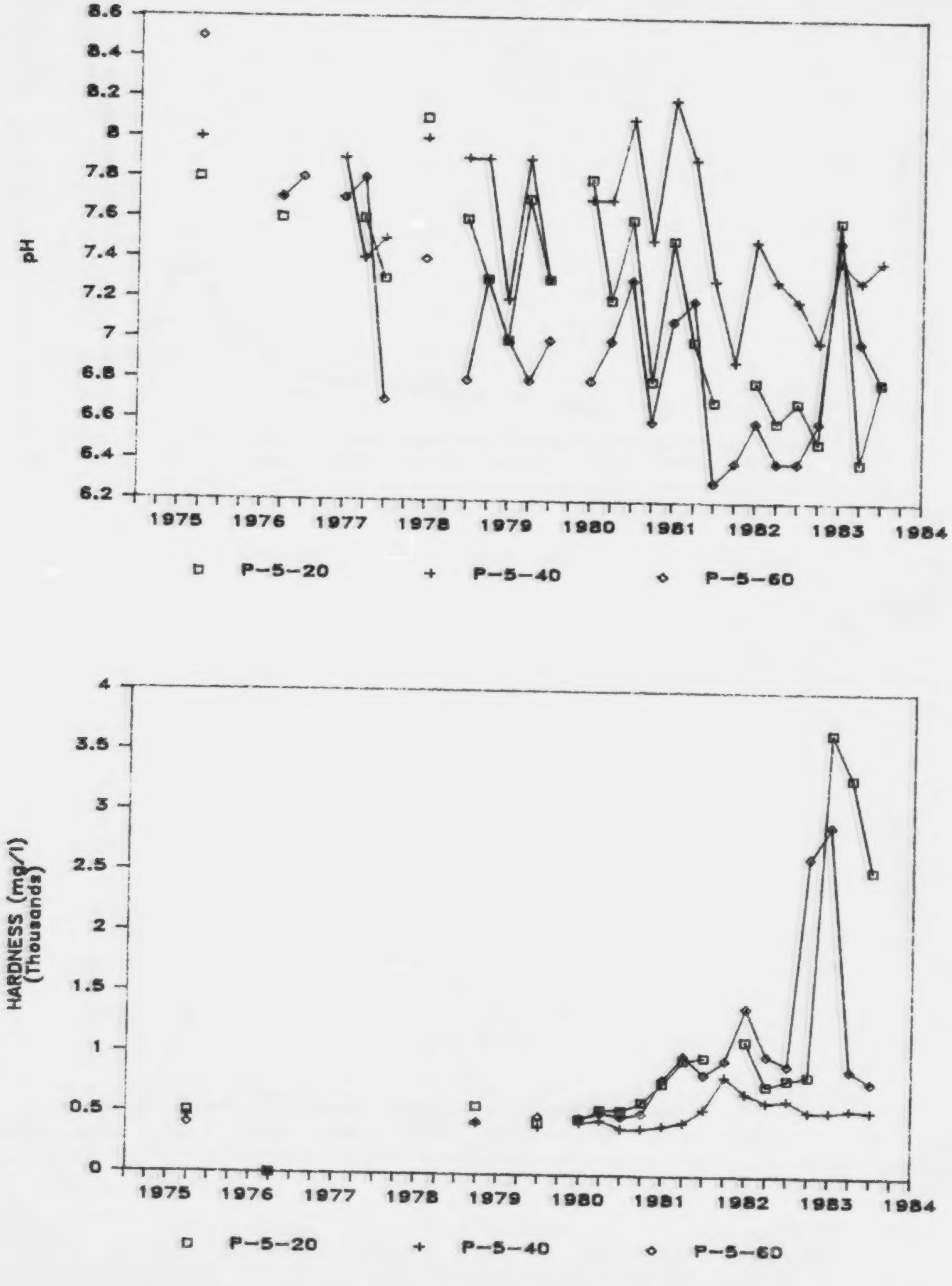


FIGURE 10: VARIATIONS OF pH AND HARDNESS IN WELL NEST P-5 AT THE WINNEBAGO COUNTY LANDFILL SITE

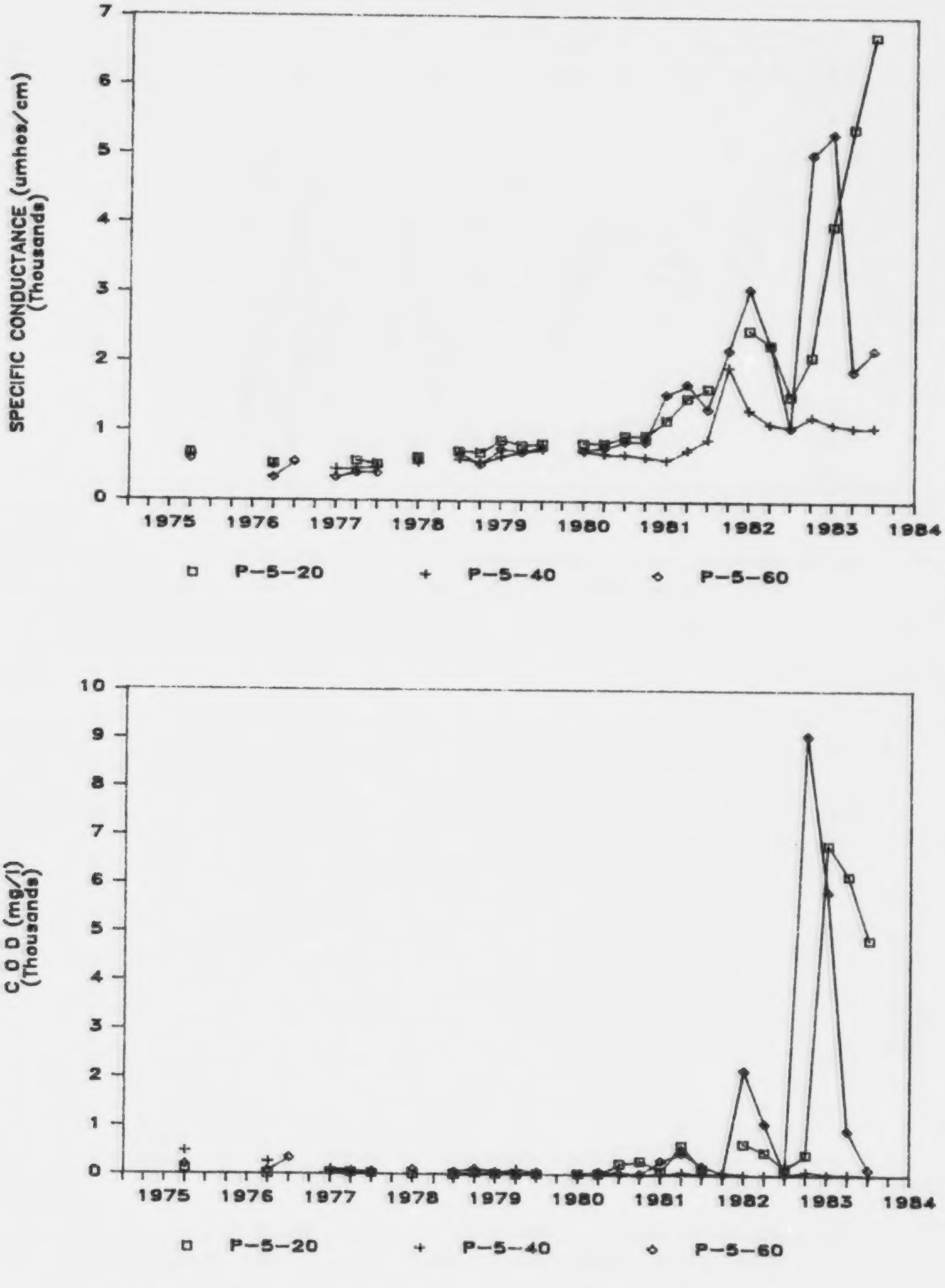


FIGURE 11: VARIATIONS IN SPECIFIC CONDUCTANCE AND CHEMICAL OXYGEN DEMAND IN WELL NEST P-5 AT THE WINNEBAGO COUNTY LANDFILL SITE

exists between the waste and surrounding saturated environment is to overexcavate and recompact the base and sidewalls of the excavation. Laboratory tests must first be conducted on representative soil samples to confirm that the on-site materials possess the qualities necessary to remain in a low permeability recompacted state.

Leachate buildup within zone-of-saturation landfills is also a major problem. This is due to factors such as: the disposal of excessive volumes of liquids or sludges, the lack of leachate management alternatives, the use of clay soils for daily and intermediate cover and inadequate removal of leachate.

To limit leachate buildup in a new or expanding zone-of-saturation landfill the DNR's Bureau of Solid Waste Management now requires granular drainage blankets to be placed on the recompacted base and sidewalls of the facility. The performance of these drainage blankets has not been thoroughly documented. However, investigations performed at the Weyerhaeuser sludge landfill (#2873) illustrate that a 1 foot (0.3 m) thick sand blanket is effectively maintaining "dry base" conditions. A drainage analysis performed during the design of the northern most trench predicted that a base slope of 1% to 1.5% and a maximum leachate flow distance of 100 feet (30.5 m) would maintain leachate head levels within the sand blanket. A follow-up evaluation of the collection system was undertaken prior to ultimate closure of the site. This evaluation revealed that while the sludge is saturated to within 1 foot (0.3 m) of the fill surface, initial measurements from 4 standpipes installed into the sand blanket were dry (Table 5). This indicates a "dry base" condition is being maintained at these locations. Water induced into the standpipes shortly after installation rapidly dissipated documenting that the sand blanket has a permeability of 1×10^{-3} cm/sec or greater. These results are significant because they show that even though the sludge is essentially saturated the driving head on the clay base is typically less than 1 foot (0.3 m).

One of the most important variables in analyzing the efficiency of the collection system and its ability to limit leachate head buildup is the volume of leachate and ground water which must be removed. These volumes can be estimated using water balance calculations (Gee, 1983; Fenn et al., 1975; Thornthwaite and Mather, 1957) and ground water inflow equations (Cedergren, 1977). While the water balance method is generally well accepted, the assumptions upon which the predictions are based can lead to questions regarding the accuracy of the results (Kmet, 1982). Table 6 lists engineered landfills in Wisconsin for which field data on the volumes of leachate collected are summarized. The first landfills are zone-of-saturation sites and the last clay-lined sites located above the water table. Data from the clay-lined sites are presented for comparative purposes.

The available data can be used to draw some general conclusions. First, the data suggest that leachate collection volumes for zone-of-saturation sites fall in the range of 200 to 500 gallons per acre per day (1.9 to 4.7 m^3/ha/day). Two notable exceptions are Winnebago County and WMI-Omega Hills. The higher volumes at Winnebago County may be due to the acceptance of large quantities of high liquid content

TABLE 5(1)

Standpipe Number	Depth (ft)	Installation Date	Depth to Sand Layer	Water Level (Below Sludge Surface) 10-8-1	10-16-81	11-6-81	Remarks
SP-1	16.4	9-22-81	14.8	Dry	---	Dry	Dry after installation, dry after addition of 25 gallons of water.
SP-2	19.5	9-22-81	17.7	Dry	---	Dry	Dry after installation, 0.3 feet standing water after addition of 20 gallons of water.
SP-3	24.0	9-22-81	23.6	22.5	---	---	Dry after installation, 25 feet standing water after addition of 10.0 gallons of water, 1.25 feet of water 7 minutes later.
SP-4	19.3	9-22-81	18.1	18.0	---	16.0	Dry after installation, 20 feet standing water after addition of 15 gallons; 8 feet standing water after 37 minutes.
SP-5	9.1	10-8-81	23.6	Dry	4.25	1.25	Dry after installation, no water added.

(1) Wolf, T., Hermann, D. and Perpich, W., 1981. "Design Report and Place of Abandonment, Trench 2, Weyerhaeuser Co. Paper Mill Sludge Disposal Landfill." STS Consultants, Ltd., Green Bay, Wisconsin.

TABLE 6: LEACHATE COLLECTION VOLUMES FROM WISCONSIN LANDFILLS

License #	Name	Year	Total Leachate Collected (Gallons)	Average Area Contributing To Leachate Generation (Acres)	Leachate Collected Gallons/Day	Leachate Collected Gallons/Acre/Day
2484	Outagamie County	1977	369,000	26	1,010	38.9
		1978	638,800	26	1,750	67.3
		1979	617,200	26	1,691	65.0
		1980	1,073,900	26	2,942	113.2
		1981	935,250	26	2,562	98.6
		1982	4,325,250	32	11,850	370.3
		1983	7,310,100	37	20,028	524.0
611	Winnebago County	1978	7,038,330	27	19,283	714.2
		1979	12,215,900	34	33,468	984.4
		1980	6,903,000	41.5	18,912	455.7
		1981	12,006,000	48.5	32,893	678.2
		1982	20,086,200	55.0	55,031	1000.6
		1983 (Jan.-March 14)	4,936,660	55	67,625	1229.6
1678	WMI-Omega Hills (Western System)	1978 (June 16-December)	1,083,805	33	5,446	165.0
		1979	1,848,560	33	5,065	153.5
		1980	2,196,171	33	6,017	182.3
		1981	2,530,750	33	6,934	210.1
		1982	3,454,000	33	9,463	286.8
		1983	31,942,160	33	84,513	2,651.9
1678	WMI-Omega Hills (Eastern System)	1981 (August 30-December)	247,700	10	1,998	199.8
		1982 (January-May 7)	311,500	10	2,453	245.3
		1983 (May 9-December)	5,634,630	10	22,905	2,290.5
2873	Weyerhaeuser Company	1979 (November 1-December)	638,500	5.5	10,467	1,903.1
		1980	2,461,000	7.5	6,742	899.0
		1981	640,900	7.5	1,756	234.1
		1982	676,500	7.5	1,853	247.1
		1983	799,050	7.5	2,189	291.9
1099	WMI-Metro (Western System)	1981	4,144,400	17	8,615	506.8
		1982	2,405,110	17	6,589	387.6
		1983 (January-May)	923,600	17	6,117	359.8
572	Land Reclamation	1982 (April 19-December)	215,200	10	837	83.7
		1983	1,230,000	10	3,370	334
2569	Brown County East	1977	29,900	5	82	16.4
		1978	234,550	10.5	643	61.2
		1979	505,950	17	1,386	81.5
		1980	257,255	17	705	41.5
		1981	376,801	21	1,032	49.2
		1982	592,837	24	1,624	67.7
		1983	828,007	24	2,269	94.5
2821	Eau Claire County	1979	108,040	3.5	296	84.6
		1980	671,500	7.0	1,840	262.8
		1981	926,000	7.0	1,167	166.7
		1982	824,000	10.5	2,257	215.0
		1983	1,389,000	14.0	3,805	271.8
2822	City of Janesville	1979	-0-	5.5	-0-	-0-
		1980	555	10.0	1.5	0.15
		1981	29,200	14.0	80	5.7
		1982	102,155	18.0	280	15.5
		1983	149,950	18.0	411	22.8
2895	WMI-Muskego	1981	700,070	6.5	1,918	141.0
		1982	485,000	13.6	1,315	76.9
		1983		17.1	2,380	105.3
2892	Marathon County	1981	267,224	4.5	732	162.7
		1982	577,148	9.0	1,581	175.7
		1983	1,036,470	14.0	2,840	202.8

sludge wastes, the collection of large volumes of surface water from open cells and/or inaccurate pumping data. The substantial increase in leachate volumes at the WMI-Omega Hills site is due to the extensive remedial action being undertaken. Four of the 5 clay-lined sites have values which fall in the range of 50 to 300 gallons per acre per day (0.5 to 3.1 m^3/ha/day). The City of Janesville landfill (#2822) has collected substantially less leachate, which may be due to perching of leachate on an interim clay cover layer or to a malfunction of the leachate collection system. Therefore, the data from the City of Janesville site should not be regarded as representative of actual leachate collection volumes from clay-lined sites.

Leachate collection volumes vary greatly with zone-of-saturation sites generating somewhat higher volumes then clay-lined facilities located above the water table. Although ground water inflow is important, the acceptance of large volumes of liquids and sludges is more critical in accounting for the greater collection volumes. These data, used in conjunction with a detailed water balance and ground water inflow analysis, should be used to design an efficient leachate collection and removal system.

Recommendations

A number of recommendations for limiting leachate head buildup and minimizing the potential for ground water contamination at landfills located in a saturated clay soil environment are outlined below:

Site Investigations

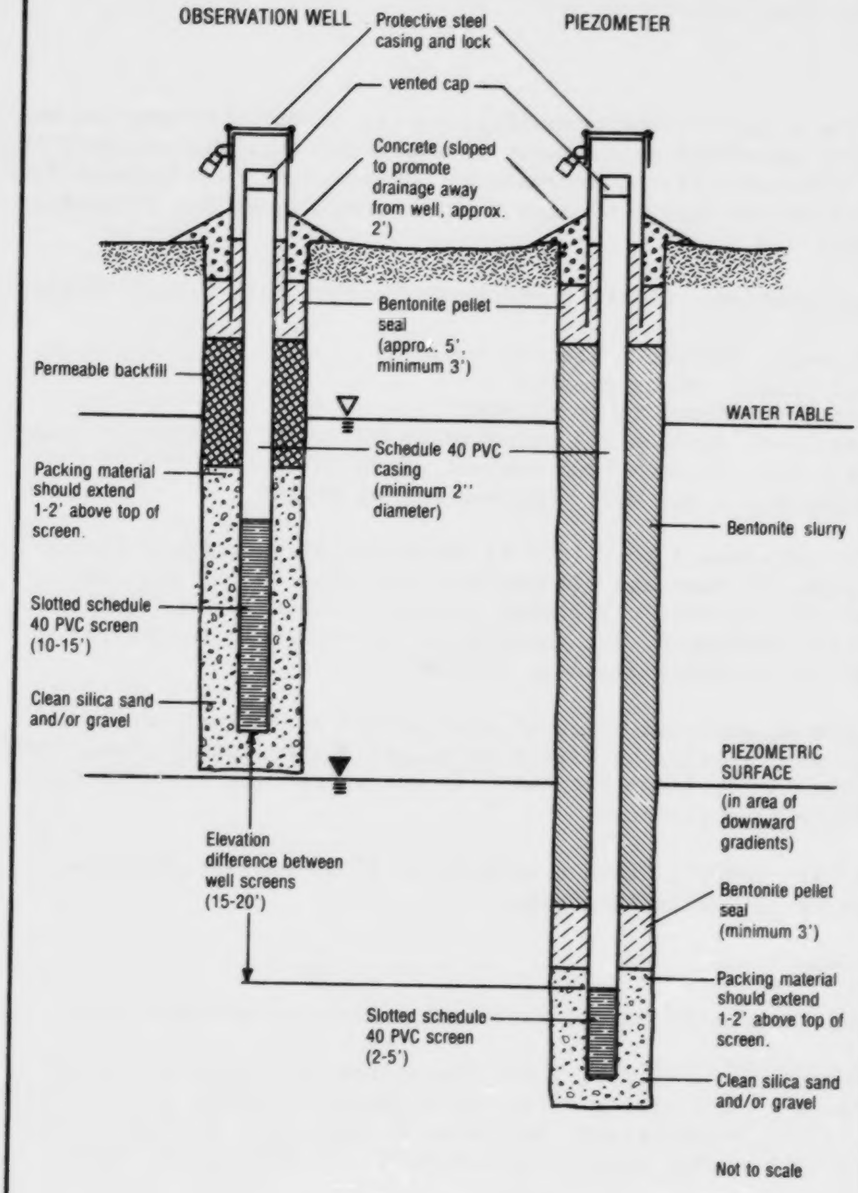
1. Drill an adequate number of soil borings on a uniform grid pattern across the site to at least 30 feet (9.1 m) below the anticipated base grade in order to characterize the clay deposits at depth.

2. Excavate backhoe pits to a depth of at least 15 feet (4.5 m) on a uniform grid pattern across the site to determine if the saturated clay deposits possess the composition necessary to limit ground water inflow to less than the rate of evaporation and to reveal any structure or permeable zones not readily identifiable in borings.

3. Install a sufficient number of water table observation wells and nested piezometers to define both the horizontal and vertical ground water flow directions. Recommended well construction details are given in Appendix A.

4. Place the well screens in the most permeable zones encountered beneath the site. Also, conduct single well response tests (slug or baildown) on all on-site wells to determine the *in situ* hydraulic conductivities.

5. Perform laboratory permeability tests on remolded samples to determine if the saturated clay soils possess the necessary composition and structure to maintain a permeability of 1×10^{-7} cm/sec or less.


OBSERVATION WELL
Protective steel casing and lock
PIEZOMETER
vented cap
Concrete (sloped to promote drainage away from well, approx. 2')
Bentonite pellet seal (approx. 5', minimum 3')
Permeable backfill
WATER TABLE
Schedule 40 PVC casing (minimum 2'' diameter)
Packing material should extend 1-2' above top of screen.
Bentonite slurry
Slotted schedule 40 PVC screen (10-15')
Clean silica sand and/or gravel
PIEZOMETRIC SURFACE (in area of downward gradients)
Elevation difference between well screens (15-20')
Bentonite pellet seal (minimum 3')
Packing material should extend 1-2' above top of screen.
Slotted schedule 40 PVC screen (2-5')
Clean silica sand and/or gravel
Not to scale

APPENDIX A: TYPICAL OBSERVATION WELL AND PIEZOMETER CONSTRUCTION DETAILS

6. Determine Atterberg limits on representative samples to define the moisture range at which the clay sediments can be properly recompacted in the field.

Site Design

1. Provide a uniform low permeability clay barrier between the waste and surrounding saturated environment by overexcavating and recompacting the base and sidewalls of the excavation. In addition to a permeability of 1×10^{-7} cm/sec or less, the clay barrier should meet the following specifications:

a) Classification: CL or CH under the Unified Soil Classification System
b) Thickness: Minimum 5 feet (1.5 m)
c) Liquid Limit: 30 or greater
d) Plasticity Index: 15 or greater
e) Compaction: Minimum 95% standard or 90% modified Proctor density
f) Grain Size: Minimum P200 content of 50% by weight
g) Clay Content (.002 mm): Minimum 25% by weight

2. Place a minimum 1 foot (0.3 m) thick granular drainage blanket over the recompacted base and sidewalls of the site to improve the efficiency of the leachate collection system. Sidewalls should be constructed at a maximum of 3:1 (horizontal to vertical) to provide stability for the granular drainage blanket.

3. Perform an analysis using an appropriate analytical model to determine the pipe spacing necessary to maintain the leachate head level within the granular drainage blanket. Do not exceed 100 feet for the maximum leachate flow distance.

4. Slope the landfill base a minimum of 2% to provide positive drainage towards the collection pipes.

Site Operations

1. Do not dispose of liquid wastes at zone-of-saturation sites.

2. In municipal landfills, limit the volume of sludge wastes to provide at least a 10:1 mixing ratio (gate yards of refuse to gate yards of sludge). Also, in municipal landfills, do not place sludges around the perimeter of the fill area or directly on the granular drainage blanket.

3. Do not use clay for daily cover on the first lift of refuse in order to prevent fines from washing onto the granular drainage blanket. Remove any intermediate clay soil cover from all subsequent lifts or use an alternate source of granular material to ensure that the waste is hydraulically connected to the underlying leachate collection system.

4. Monitor leachate collection volumes and head levels monthly so that problems can be identified early.

Conclusions

The problems identified and discussed document that Wisconsin's original 1975 guidelines for allowing landfills to be developed within the zone-of-saturation are not adequate to provide an acceptable level of ground water protection. Implementation of the recommendations outlined above should limit the potential for serious leachate head buildup and/or ground water contamination problems to develop at a landfill utilizing the zone-of-saturation design concept.

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MONITORING OF GROUND WATER CONTAMINATION FROM WASTE DISPOSAL SITES IN ALBERTA, CANADA

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ABSTRACT

There are over 1000 active and inactive solid waste disposal sites in Alberta (pop. 2.5 million, 1984). Many of these sites are located in unfavorable hydrogeological settings and consequently pose a threat to ground water quality. In light of these concerns a program has been developed to preclude the siting of waste disposal facilities in hydrogeologically unsuitable areas and to discern the effects of existing facilities on ground water quality. As part of the program, ground water monitoring systems have been installed at all regional landfills and numerous smaller waste sites.

Typically a ground water monitoring system consists of strategically located piezometers and water table wells. Water level data are used to establish the water table configuration and the horizontal and vertical ground water gradients. Quantitative and qualitative hydraulic conductivity values are obtained for various geologic materials using a variety of techniques.

Ground water samples are collected for chemical analysis using a variety of methods contingent upon field conditions and specific sample requirements. Monitoring wells are developed prior to sampling and a number of field measurements are conducted at the time of sampling. In most cases, samples are collected for routine inorganic and trace metal analysis and various organic indicators. Additional samples are collected where exotic pollutants are suspected.

Two summarized case studies underscore the importance of a properly installed ground water monitoring system. Ground water monitoring data are essential for assessing the impact of a waste site on ground water quality, in assessing the validity of hydrogeological landfill siting criteria and in devising appropriate amelioration procedures, if necessary.

INTRODUCTION

Prior to 1981, refuse disposal had been given a low priority in Alberta. As a result many waste sites violate Provincial Board of Health regulations. Infractions of these regulations were tolerated because of the political difficulties in dealing with small communities and pending the approval of regulated sites. The problem could have been mitigated by using sanitary landfill techniques involving daily covering compacted waste with soil. However the costs involved in a sanitary landfill operation are often beyond the financial means of small municipalities.

At present the province generates approximately 5.5 million tons (2.5 million tonnes) of solid waste annually. Landfilling is used almost exclusively due to Alberta's small population and abundant land. To the end of 1982 there were over 1000 active waste disposal sites (MacLaren Plansearch Lavalin, 1982). The majority of these sites are located in the southern half of the province, in an arcuate band coinciding with the major population centers. Many of the disposal sites have become open dumps producing problems of odor, blowing paper, vectors, as well as surface and ground water contamination.

Solid Waste Disposal in Alberta

Before discussing ground water monitoring, it is important to understand the various types of waste disposal methods in Alberta. This background knowledge is important since different facilities require different levels of ground water monitoring. A landfill sited in a hydrogeologically favorable environment would require a reduced level of ground water monitoring than a landfill in a poor hydrogeological setting.

Solid waste disposal sites in Alberta can be categorized into three types: open dumps, modified landfills and regional landfills. Open dumps, which are generally uncontrolled, involve filling trenches with refuse, with little consideration for environmental protection. This method of waste disposal has been prevelant in the past. Fortunately, because the most populated area of Alberta is in a semi-arid climate and is underlain by geologic materials of low permeability, ground and surface water contamination has been minimal.

Modified landfilling involves filling trenches with refuse, compaction of refuse and applying a final cover of soil. These facilities are usually controlled, more judiciously sited, and as a result are more environmentally sound than open dumps.

Regional landfills are operated as sanitary landfills whereby refuse is covered daily with compacted soil. Hydrogeological evaluations are conducted on all candidate sites. These measures afford the regional landfill a high level of ground water quality protection.

The regional landfill concept was developed by Alberta Environment to discourage the use of small poorly managed sites. Regional landfills, which are largely funded by the Alberta Government, are utilized jointly by two or more municipalities. In order to be eligible for financial assistance a regional landfill should serve a region of two or more municipalities with the population between 5,000 and 60,000. Edmonton and Calgary, the two largest cities in Alberta which generate alomost half of the waste, are excluded from the program since they operate their own sanitary landfills. At present there are 12 regional landfills in Alberta with an ultimate goal of approximately 20.

Besides the obvious economic benefit for small municipalities who cannot afford proper waste disposal, there are numerous other advantages to the regional landfill concept. Since each site must meet stringent hydrogeological criteria, ground water contamination is minimized. A ground water monitoring system is installed at each regional landfill facility to monitor the long term impact of the landfill on ground water quality.

GROUND WATER MONITORING

Ground water monitoring, which is conducted by the Earth Sciences Division of Alberta Environment, is essential for assessing landfill design criteria and for monitoring ground water quality changes which may result from a landfill. Specifically, the purpose of monitoring is to establish:

(1) the water table configuration and annual water table fluctuation;
(2) the hydraulic conductivity of geologic materials;
(3) the ground water chemistry in the vicinity of the landfill; and
(4) the extent of any contamination and composition of leachate.

Full-time monitoring activities are presently restricted to the regional landfills. To formulate an orderly approach to monitoring activities, a schedule is compiled annually. Adherence to the schedule is contingent upon numerous factors including the weather, availability of manpower and equipment and priority projects.

Numerous requests are received annually to investigate reports of ground water contamination from smaller waste sites such as modified landfills or open dumps. Alberta Environment, installs monitoring wells, collects ground water samples for chemical analysis and compiles a preliminary assessment of the problem. If continued monitoring is required at the site, local individuals or the landfill operator are instructed on ground water monitoring procedures.

Test Drilling and Monitoring Well Installation

A thorough understanding of shallow geology is essential before installing a ground water monitoring system. To this end, published information is reviewed and shallow test holes are drilled to obtain detailed information about shallow geologic conditions. Lithologies are described in the field and grab samples are collected for more detailed analysis.

An auger rig, with solid and hollow-stem capabiltiy, is most suitable for drilling shallow unconsolidated deposits in Alberta. This method is inexpensive, yields good quality samples, and relatively undisturbed cores can be collected using shelby tubes. When consolidated materials are encountered or when the depth of drilling exceeds 100 ft. (30 m) the auger system is ineffective and the rotary system is employed. Air-rotary is most often used since this method does not contaminate the formation with drilling fluid and facilitates idenfication of the saturated zone.

At each landfill site at least one test hole is advanced to 200 ft. (61 m) to provide deep geologic information. A geophysical well log is routinely run to aid in the lithologic description.

Piezometers and water table wells are subsequently installed in selected test holes. Monitoring wells are constructed using 2 in. (5 cm) i.d. PVC pipe. Typically, monitoring wells are installed up-gradient, in, and down-gradient from the disposal site. Ground water from wells located up-gradient from the disposal site represent background water quality. Ground water from wells completed in and down-gradient from the refuse are used to establish the chemical characteristics of leachate and the extent of any ground water contamination.

Hydraulic Conductivity Methodology

Single well response tests are conducted on all piezometers to obtain in-situ hydraulic conductivity measurements for various geologic materials beneath the site. The rising head method is used most frequently, with manual measurement of the water level using an electric sounder. For permeable materials (i.e > 21 gal/d/ft (10^{-3} cm/s)) the response is too rapid to be measured manually. In these instances an electric transducer coupled to a strip-chart recorder is used to record water levels.

Qualitative estimates of hydraulic conductivity are frequently obtained by grain size or sieve analysis. The Hazen formula (Hazen, 1983) is most often used to estimate hydraulic conductivity from these data.

Ground Water Sampling

Each well is developed prior to sampling to ensure abstracted ground water is representative of formation water. Wells completed in low permeability materials are evacuated a minimum of five times. A 1.75 in (4.4 cm) i.d. submersible pump is used to abstract water from wells completed in permeable deposits. Bailing or pumping continues until the electrical conductivity of abstracted water is constant.

Ground water samples are collected using a variety of methods contingent upon field conditions. A peristaltic pump is most frequently employed because of its portability and convenience. In instances where the depth to water exceeds 30 ft. (9 m) or when the well is completed in highly permeable deposits, a stainless steel submersible pump is used. A teflon bailer is used when a well is not readily accessible or has a small diameter. Samples are filtered in the field using a 0.45 µm membrane filter. The temperature, electrical conductivity and pH of the sample are recorded in the field. All sampling and filtering equipment are thoroughly cleaned after each sampling to minimize cross-contamination.

The frequency of ground water sampling is dependant upon the contamination potential of the site and stage of monitoring. Water samples are collected for chemical analysis twice or more times annually at a new landfill or at one in which there is potential for contamination. When the potential for contamination is reduced, water samples are collected from selected wells once annually.

Information about the type of waste in the landfill is acquired before ground water sampling commences. This information is essential for determining the number of samples required and most importantly the type of analysis. Routinely, at a new landfill, ground water samples are collected for inorganic and trace metal parameters. In addition samples are collected for analysis of organic indicators. A listing of parameters analyzed and preservation techniques used in routine sampling is contained in Table 1. After the first year, if it is apparent that there is no ground water contamination, the chemical analysis is limited to key indicators of leachate contamination such as total dissolved solids, chloride, iron and total organic carbon. When industrial wastes are known to exist in a landfill, additional samples are collected for polychlorinated biphenyls (PCBs), mercury, and phenol analysis and a hydrocarbon scan. Where extensive organic contamination is suspected a sample or samples is (are) collected for Gas Chromatography-Mass Spectrometry (GC/MS) scan to indentify priority pollutants.

Table 1
Sampling Procedures and Preservation Techniques

Chemical Parameter	Method of Preservation
Inorganic	
Cl, SO_4, HCO_3, NH_4	refrigerate
Na, K, Ca, Mg	
Organic Indicators	
COD	acidify with 5 ml H_2SO_4 and refrigerate
TOC	freeze
Metals	
Cu, Fe, Mn, Pn, Zn	acidify with 5 ml (1:1) HNO_3 and refrigerate

Temporal and natural spatial variations in shallow ground water chemisty must be considered when interpreting analytical results. To make the correct interpretation regarding the relative impact of a waste facility on ground water quality it is important to differentiate variability in ground water quality due to the waste facility from variability in ground water quality due to other factors. Factors affecting spatial-temporal variations in ground water have been outlined by Keith, et.al. (1982). These factors include well construction, development, geology, hydraulic conditions, sampling time and sample analysis. Samples are collected at the same time annually, by the same personnel and are analyzed by the same lab to reduce the effects of some of these factors. Spatial ground water quality variations due to geology are significant, particularily in Alberta where geologic materials are characterized by extreme heterogenity and anisotropy.

FUTURE PROGRAMS

A substantial amount of water level and water quality data are being generated by landfill monitoring activities. To cope with the increased load, computer storage in planned which will significantly reduce labor costs by simplifying data manipulation and updating.

Recently the province completed a survey of the environmental sensitivity of waste disposal sites in Alberta. The survey identified approximately 400 sites of concern which warrant further study. To conduct a traditional testhole drilling - sampling study at each of these sites was not found to be economically feasible. The possibility of using geophysicial techniques for quick, first-round evaluations of these sites was explored. Geophysical techniques considered included: ground penetrating radar, electromagnetics, resistivity, seismic refraction, magnetometry and metal detectors. The electromagnetic (EM) technique was found to be the most economical and suitable technique for climatic and geological conditons in Alberta. A study was conducted to test EM in different geologic conditions. Five landfills, located in different geological environments were chosen as test sites. A detailed report of study findings will be presented in a future paper. Briefly, results corroborate results from other such studies: the method is useful for detecting shallow ground water contamination, particularly in sandy soils. EM was least useful in areas underlain by low permeability clays or in areas of high topographic relief. It was concluded that the technique is a useful tool. However, follow-up drilling was required to confirm EM results.

CASE STUDIES

Two summarized case studies are presented as examples of ground water monitoring systems at landfills in Alberta. The first illustrates the importance of understanding the geology of a site before interpreting ground water chemistry data and the second demonstrates the usefulness of geophysics in assessing ground water contamination.

Wetaskiwin Regional Landfill

The landfill site occupies 0.5 mi^2 (1.3 km^2) of partly tree covered terrain in south-central Alberta. It is operated as a sanitary landfill accepting primarily municipal waste.

Surficial deposits covering the area consist predominantly of silty clay till having a hydraulic conductivity in the order of 10^{-3} gal/d/ft (10^{-7} cm/s). Eolian deposits which form sand dunes, partly overlie the till. Non-marine bedrock comprised of grey, feldspathic, clayey sandstone, grey bentonitic mudstone and carbonaceous shale subcrops in the area.

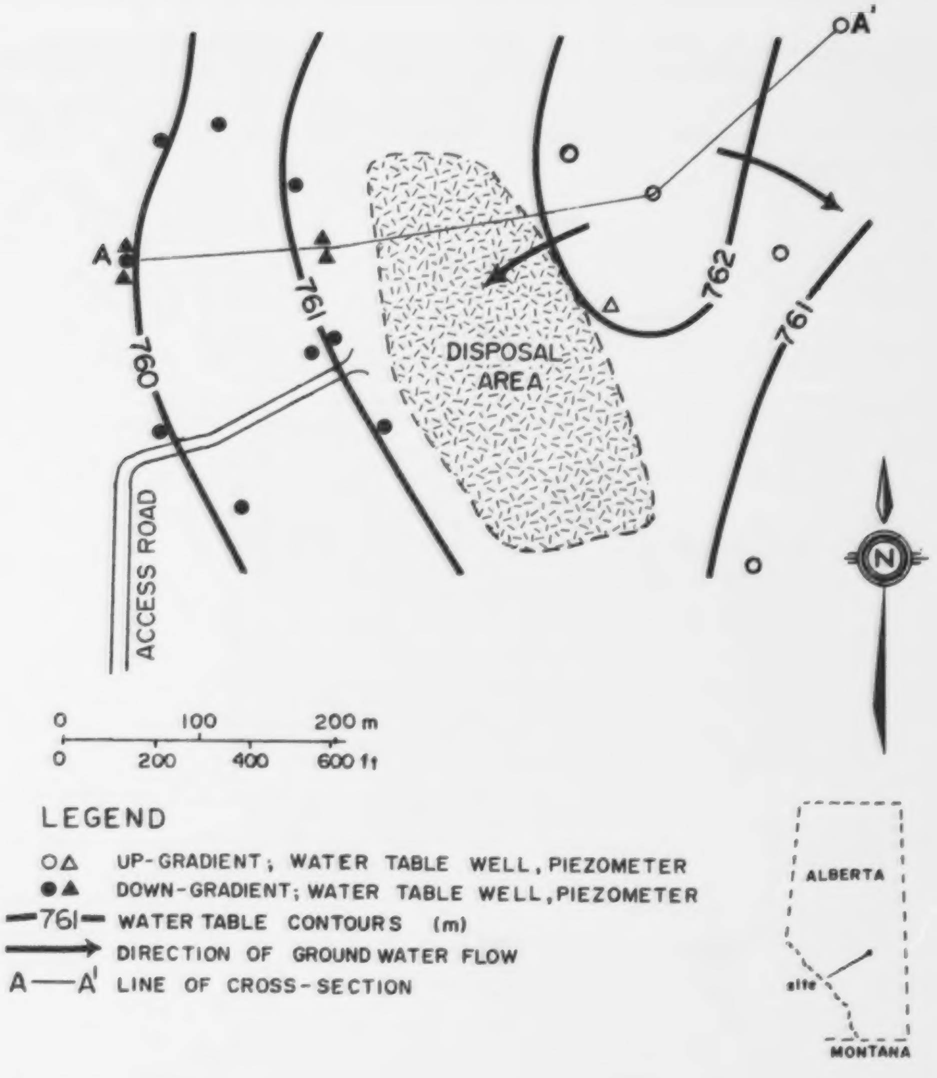
A'
A
760
761
762
761
DISPOSAL AREA
ACCESS ROAD
N
0 100 200 m
0 200 400 600 ft
LEGEND
○△ UP-GRADIENT; WATER TABLE WELL, PIEZOMETER
●▲ DOWN-GRADIENT; WATER TABLE WELL, PIEZOMETER
—761— WATER TABLE CONTOURS (m)
DIRECTION OF GROUND WATER FLOW
A—A' LINE OF CROSS-SECTION
ALBERTA
site
MONTANA

Figure 1. Site plan and water table contours - Wetaskiwin regional landfill.

A detailed plan showing the locations of all piezometers and water table wells is presented in Figure 1. Regionally, the direction of ground water flow is toward the northeast (Ceroici, 1979). Water level data indicate that the regional ground water system in the landfill area is modified. The water table beneath the sand dunes is mounded, forming a ground water divide. The hydraulic head decreases with depth indicating the area is located in a ground water recharge area.

Shallow ground water in the study area is dominantely one of three types:

I Calcium/magnesium-bicarbonate type with a total dissolved solids (TDS) concentration less than 250 mg/L.

II Calcium/magnesium-bicarbonate/sulphate to sodium-bicarbonate type with a TDS concentration ranging from 500 to 1300 mg/L.

III Sodium-bicarbonate type with a TDS concentration ranging from 700 to 900 mg/L.

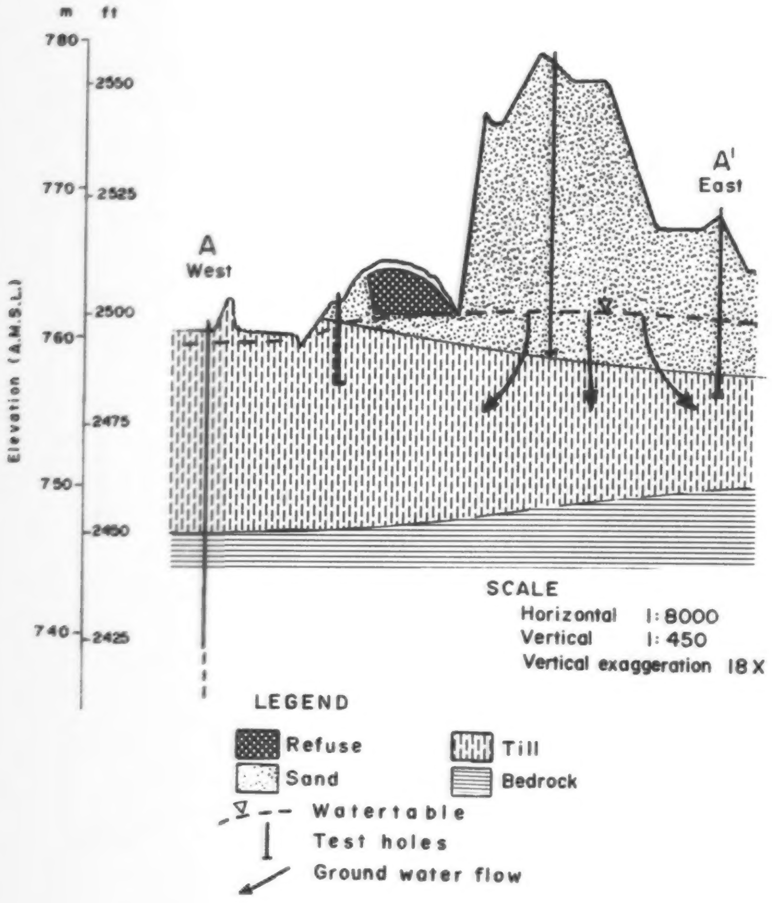
m
ft
780
2550
770
2525
2500
760
2475
750
2450
740
2425
Elevation (A.M.S.L.)
A
West
A'
East
SCALE
Horizontal 1:8000
Vertical 1:450
Vertical exaggeration 18X
LEGEND
Refuse
Sand
Till
Bedrock
Watertable
Test holes
Ground water flow

Figure 2. Geologic cross-section.

Type II ground water, collected from wells located down gradient from the disposal site was of poor quality relative to type I ground water from wells located upgradient from the landfill. Initially it was suspected that the landfill had contaminated shallow ground water. A subsequent analysis of local geologic conditions indicated that spatial variations in shallow ground water chemistry are significantly influenced by the geology. Figure 2 and 3 illustrate the stong correlation between ground water chemistry and geology. To establish the degree of ground water contamination, additional wells were installed in the vicinity of the disposal area. Subsequent ground water sampling and analysis indicated that ground water contamination was restricted to the area immediately beneath the refuse. This is corroborated by hydraulic conductivity data which indicate that the low hydraulic conductivity of till would preclude significant leachate migration.

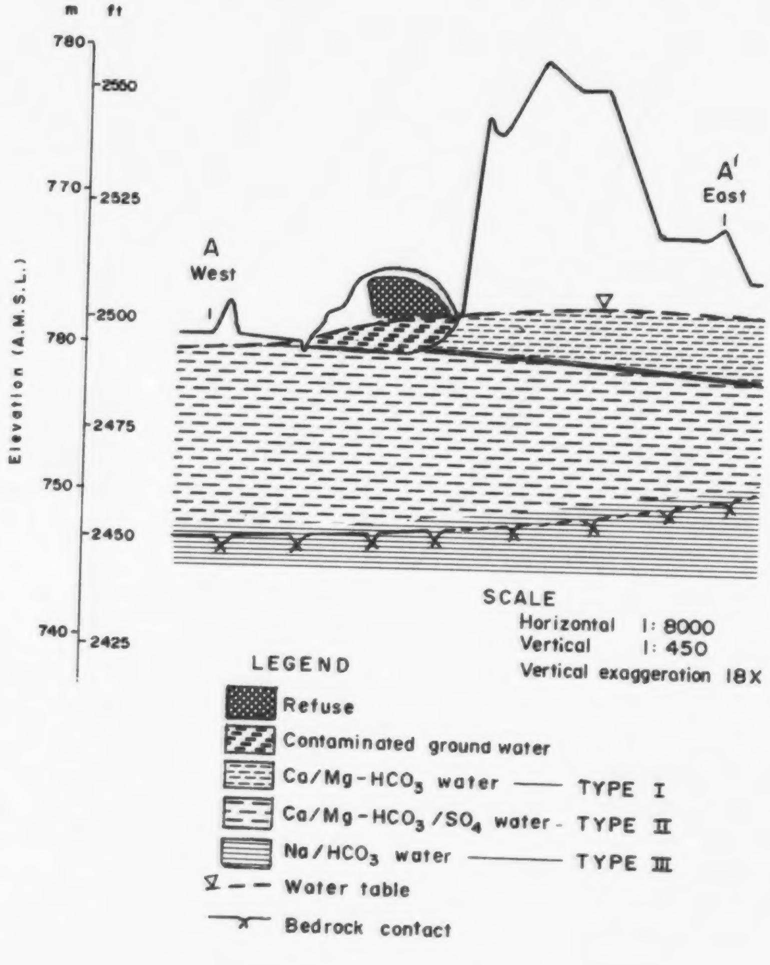
m
ft
780
2550
770
2525
760
2500
2475
750
2450
740
2425
Elevation (A.M.S.L.)
A
West
A'
East
SCALE
Horizontal 1: 8000
Vertical 1: 450
Vertical exaggeration 18X
LEGEND
Refuse
Contaminated ground water
Ca/Mg-HCO3 water —— TYPE I
Ca/Mg-HCO3/SO4 water - TYPE II
Na/HCO3 water —— TYPE III
Water table
Bedrock contact

Figure 3. Cross-section showing distribution of groundwater types.

Hardisty Waste Disposal Site

The Hardisty waste site, which is approximately 2.5 ac (1 ha) in size, is located in east-central Alberta. In 1982 the site, which contains primarily domestic refuse, was abandoned due to suspected ground water contamination.

The shallow geology in the vicinity of the landfill consists of 0.8-1.5 ft (2.7 - 5m) of coarse-grained sand overlying clayey till. Water level data indicate that the predominant direction of shallow ground water flow is toward the northeast (Figure 4). Ground water flow is most active in the sand owing to the high permeability contrast between the sand and till.

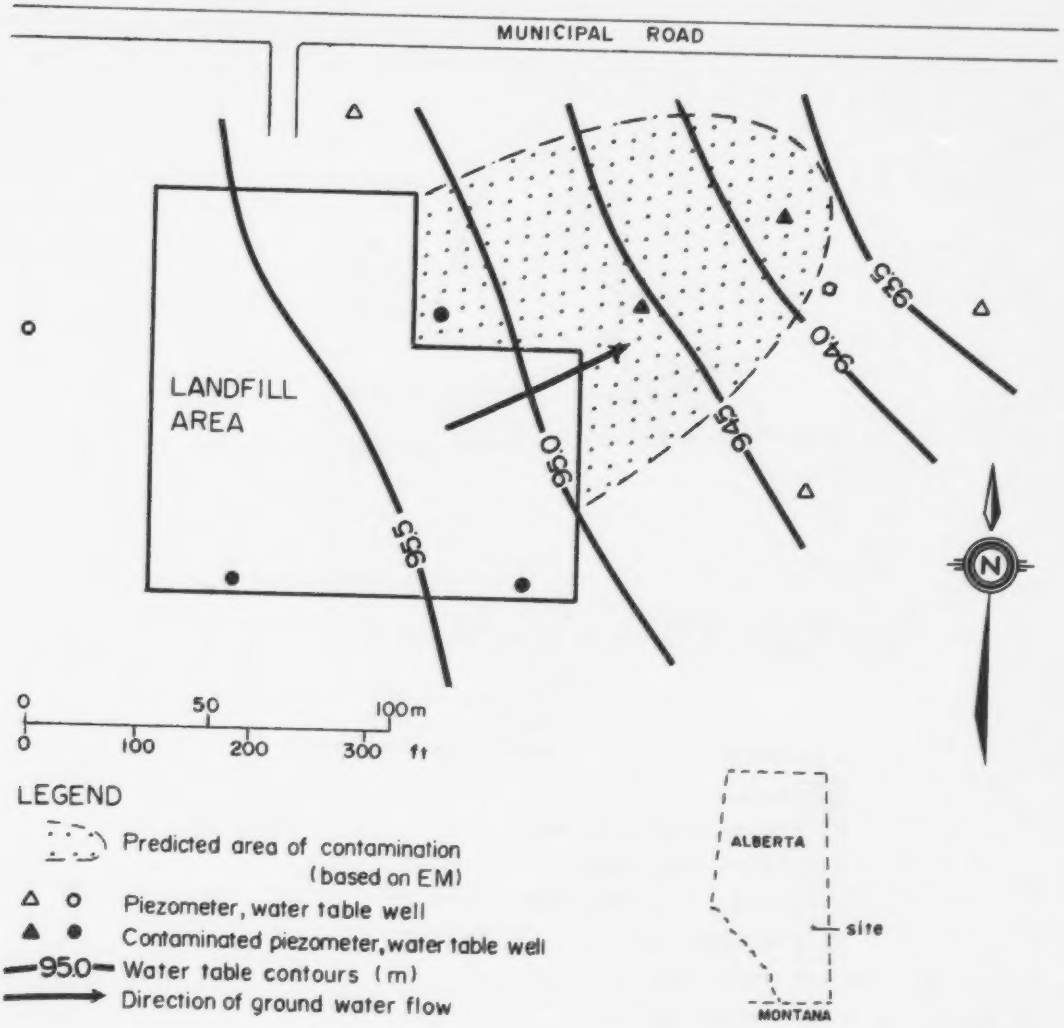
MUNICIPAL ROAD
LANDFILL AREA
955
950
945
940
935
N
0 50 100m
0 100 200 300 ft
LEGEND
Predicted area of contamination (based on EM)
Piezometer, water table well
Contaminated piezometer, water table well
950 Water table contours (m)
Direction of ground water flow
ALBERTA
site
MONTANA

Figure 4. Site plan and predicted area of groundwater contamination - Hardisty landfill.

An electromagnetic (EM) survey was employed to establish the boundaries of the refuse block and to detect ground water contamination. Different electrode spacings were used to obtain conductivity readings from different depths in order to obtain a three-dimensional representation. EM data effectively delineated the refuse area and indicated that a leachate plume was emanating from the refuse area to the northeast. Piezometers were installed to establish shallow ground water chemistry and confirm contamination (Figure 4). Excessive sulfate, chloride and trace metal concentrations in ground water from wells located down gradient, confirmed that shallow ground water was contaminated.

The EM survey proved to be a valuable tool. It facilitated rapid delineation of the refuse block and ground water contamination. Further, it aided in the placing of piezometers therby reducing field costs.

CONCLUSIONS

The amount of waste generated in Alberta is increasing annually due to increases in population and industrial activity. To accomodate the waste and to mitigate the impact of waste disposal facilities on ground water resources, it is essential to employ site selection criteria. Further, to monitor the long term effects of landfill facilities on ground water quality, it is essential ground water monitoring systems be installed.

Alberta has made a positive move toward protecting ground water quality by establishing the regional landfill concept. The number of small scale dumps is decreasing since more municipalities can afford proper waste disposal. Regional landfills are operated as sanitary landfills where refuse is compacted daily and covered with soil.

Ground water monitoring systems are installed at all regional landfills and selected smaller facilities. These systems consist of strategically located piezometers and water table wells which are sampled at a frequency contingent upon the contamination potential of the site. Monitoring activities are organized annually by compiling a schedule.

Present and future monitoring activities will put a stress on present manpower and funds. To cope with this difficulty, inovative techniques are being sought to streamline monitoring activities. Computer storage of ground water monitoring data is planned to cope with the increased amount of data. Geophysical techniques are being explored as methods to quickly evaluate ground water contamination from waste facilities.

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BIOGRAPHICAL SKETCH

W.J. Ceroici is presently a Hydrogeologist with the Earth Sciences Division of Alberta Environment. He received a B. Sc. (Honors) in Geology from the University of Alberta in Edmonton, Alberta, Canada and an M. Sc. in Earth Sciences from the University of Waterloo in Waterloo, Ontario, Canada. In the past, he has been employed by the Alberta Research Council and Monenco Consultants Limited as a Senior Hydrogeologist. He is currently involved in numerous ground water contamination studies, particularly from waste disposal sites.

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VERTICAL MOVEMENT OF GROUND WATER UNDER A LANDFILL, ANCHORAGE, ALASKA

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Anchorage, Alaska

Abstract

A thorough review of existing ground-water information may, in some cases, be adequate to estimate rates of migration of pollutants. Analysis of data from well-performance tests and from hydrologic-data stations near a landfill in Anchorage, Alaska, indicates that pollutants migrating downward toward a confined aquifer that supplies water to three municipal wells near the landfill do not pose an imminent threat to the water supply.

The hydrostatic relationship, total head = gravity head + pore pressure, was used to estimate that gradients through two confining units are 1.6 and 0.033. An analysis of extensometer data for elastic compaction of sediments indicated that specific storage of the sediments is 2.3×10^{-6} reciprocal feet (7.54×10^{-6} reciprocal meters). An analysis of several well-performance tests using the extensometer results and Hantush's leaky-aquifer theory indicates that vertical hydraulic conductivities of two confining units are 10^{-2} and 2×10^{-2} feet per day (3×10^{-3} and 6×10^{-5} meters per day). The values of vertical hydraulic conductivity and vertical gradients were used with Darcy's law and theoretical solute transport curves to estimate that incipient entry of pollutants to the aquifer may occur about 80 years after leachate began migrating downward. Breakthrough of "full-strength" leachate probably will not occur prior to 250 years after downward migration began.

The analysis helps alleviate some concerns that pollution of municipal wells is imminent. However, because the errors in estimating hydraulic conductivities may be as great as a factor of three, the analysis should not be used as justification to discontinue monitoring migration of the leachate.

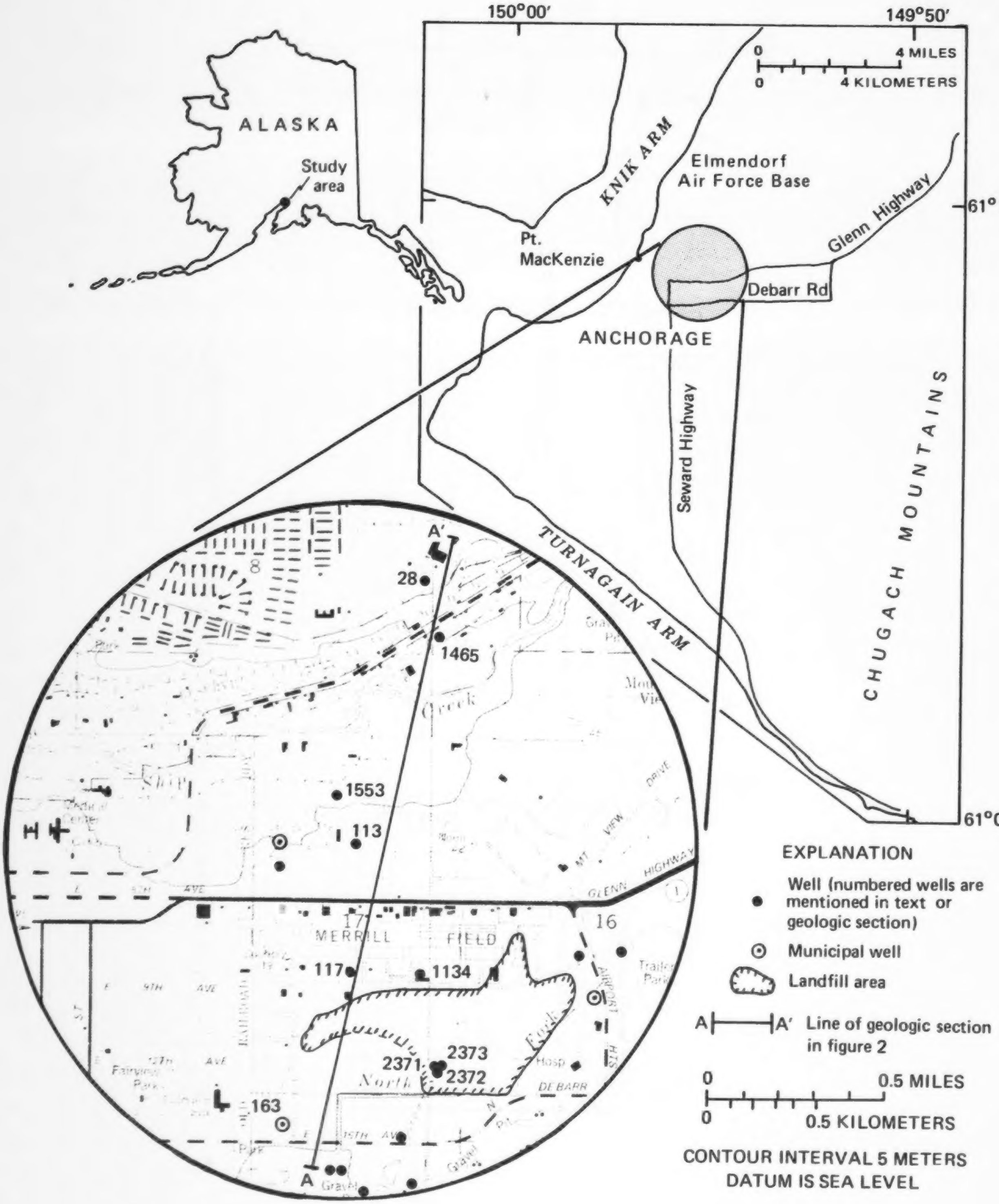
Introduction

The Merrill Field solid-waste landfill at Anchorage, Alaska, (fig. 1) has been utilized for refuse disposal since about 1950. When the site is fully utilized, approximately 150 acres (61 ha) will be covered with soil and refuse to an average thickness of about 30 ft (9 m). Some of the refuse has been buried below the water table, thereby creating an environment in which the refuse in continuously leached.

Three municipal-supply wells withdraw water from a lower confined aquifer 200 to 300 ft (60 to 90 m) below the land surface in the vicinity of the landfill. Other wells near and downgradient from the landfill obtain water from an upper confined aquifer 75 to 190 ft (23 to 58 m) below land surface. The purpose of this study was to use existing aquifer-test and water-level data to estimate the hydraulic gradients and hydraulic properties that affect the rate at which the polluted shallow ground water migrates downward from the landfill to the aquifers that supply the municipal wells.

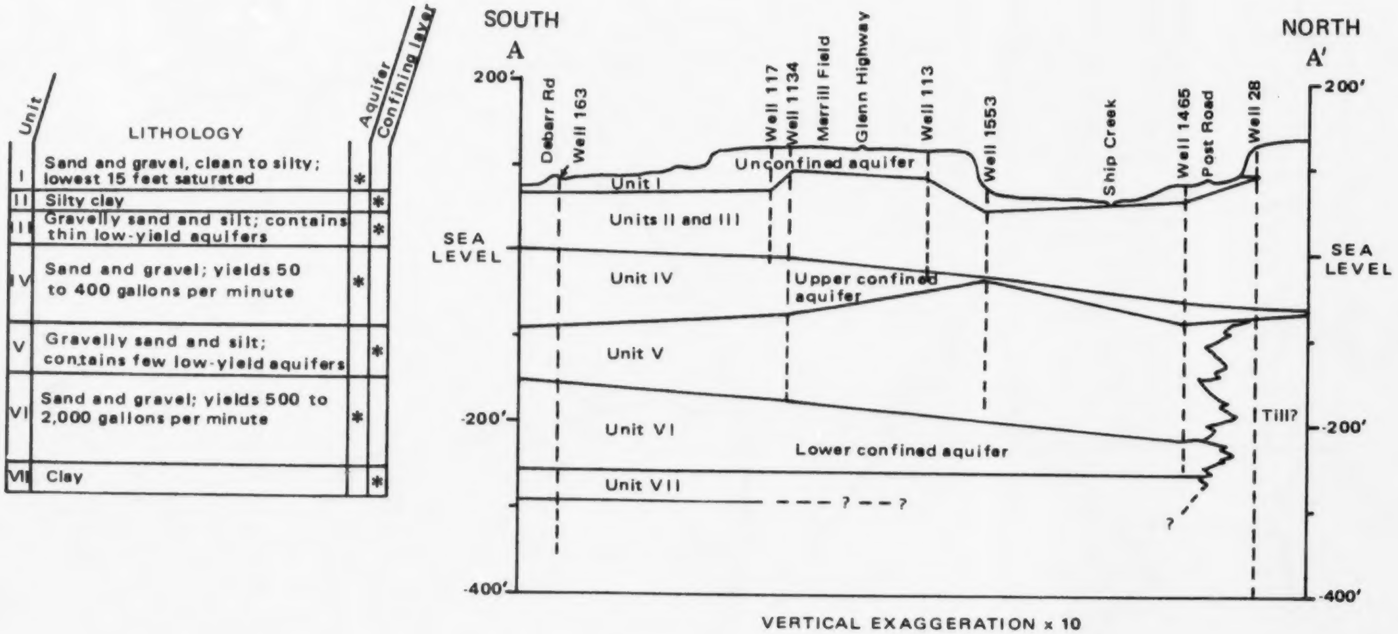
Geology

The landfill area is underlain by a complex sequence of stratified glacial, fluvial, and lacustrine sediments which the author generalized into seven units (fig. 2). The saturated part of Unit I is the unconfined aquifer. Units III and V are lithologically similar, are generally of low permeability, and act as confining units. However, both contain a few thin stringers of sand and gravel that are low-yield aquifers. The hydraulics analysis in this report is based in part on the premise that the vertical



ALASKA
Study area
150°00′
149°50′
61°15′
61°00′
0
4 MILES
0
4 KILOMETERS
KNIK ARM
Elmendorf Air Force Base
Glenn Highway
Pt. MacKenzie
Debarr Rd
ANCHORAGE
Seward Highway
TURNAGAIN ARM
CHUGACH MOUNTAINS
A′
28
1465
1553
113
117
1134
2371
2373
2372
163
A
MERRILL FIELD
17
16
8
GLENN HIGHWAY
MT. VIEW DRIVE
DEBARR
North
Hosp
Trailer Park
Fairview Park
Gravel
AIRPORT HTS
EXPLANATION
Well (numbered wells are mentioned in text or geologic section)
Municipal well
Landfill area
A — A′ Line of geologic section in figure 2
0
0.5 MILES
0
0.5 KILOMETERS
CONTOUR INTERVAL 5 METERS
DATUM IS SEA LEVEL

Figure 1.--Location of study area and wells.


SOUTH
A
NORTH
A′
200′
SEA LEVEL
-200′
-400′
Deberr Rd
Well 163
Well 117
Well 1134
Merrill Field
Glenn Highway
Well 113
Well 1553
Ship Creek
Well 1465
Post Road
Well 28
Unconfined aquifer
Unit I
Units II and III
Unit IV
Upper confined aquifer
Unit V
Unit VI
Lower confined aquifer
Unit VII
Till?
?
VERTICAL EXAGGERATION x 10

Unit	LITHOLOGY	Aquifer	Confining layer
I	Sand and gravel, clean to silty; lowest 15 feet saturated	*	
II	Silty clay		*
III	Gravelly sand and silt; contains thin low-yield aquifers		*
IV	Sand and gravel; yields 50 to 400 gallons per minute	*	
V	Gravelly sand and silt; contains few low-yield aquifers		*
VI	Sand and gravel; yields 500 to 2,000 gallons per minute	*	
VII	Clay		*

Figure 2.--Generalized stratigraphic column and geologic section at Merrill Field landfill.

hydraulic conductivities and storage properties of Units III and V are similar. Units IV and VI are the principal aquifers and are commonly referred to as the upper and lower confined aquifer.

Hydraulic Characteristics of the Sediments

Vertical Gradients

Before 1958, pumping from the confined aquifers was insignificant. The potentiometric surface of the upper confined aquifer was higher than the water table, the gradient was upward, and thus there was no potential for downward migration of pollutants. Since 1958, the potentiometric surface has been lower than the water table, thereby producing a downward gradient. This condition is shown (for 1974) by the hydrographs in figure 3. The gradient between the upper and lower confined aquifers may be upward or downward, depending on distribution of pumping from them. The potentiometric surface of the upper confined aquifer under the landfill reached a record low level of 39 ft (12 m) above sea level in April 1975.

During much of the time since 1971, the potentiometric surface of the upper confined aquifer has been below the base of Unit II. When well 2371 was drilled in July 1974, the potentiometric surface of the upper confined aquifer (Unit IV) was about 20 ft (6 m) below the base of the clay of Unit II. The sediments at the top of Unit III yielded no water to the well during drilling. A diagram of the vertical gradients under the landfill in July 1974 (fig. 4), is based on the following assumptions:

1. Pore pressures in aquifers increase hydrostatically with depth. This assumption is strictly true only if there is no vertical flow in the aquifers, but it is approximately true if head lost by vertical flow in the coarse materials is very small.

2. Where pore pressure is negative at the boundary of Units II and III, the maximum tension is -1.5 ft (0.46 m). E. P. Weeks (written commun., 1976) determined a tension of -1.5 ft (0.46 m) under sim-

ilar conditions.

3. Units are homogeneous, so pore pressures change linearly within the units.

Total head was calculated as the sum of the gravity head, which decreases directly with depth, and the pore pressure. Water levels were plotted for wells 2371, 2372, and 2373 on figure 4. The slope of the pore-pressure curve in Unit III, which was calculated on the basis of the water level in well 2372, shows that the pore pressures reach zero at 9 ft (2.7 m) below the base of Unit II. This is consistent with the observation during drilling that the upper sediments in Unit III yielded no water. The diagram indicates that the total-head gradient through Unit II (1.6) is much greater than through the saturated part of Unit III (0.033). These conditions are considered to be typical for periods when the municipal wells are pumping.

At steady flow the ratio of the hydraulic conductivities of Units II and III (K_{III}/K_{II}) is equal to the inverse of the ratio of the gradients through the units (1.6/0.03). That ratio indicates that the hydraulic conductivity of Unit III is about 50 times that of Unit II.

Extensometer Analysis

An extensometer in well 1134 (fig. 5) measures the compaction of sediments as the pore pressures are reduced by pumping. Readings from the shallow extensometer reflect seasonal changes in the altitude of the land surface that are caused by freezing or thawing of the ground. These readings are subtracted from the compaction values calculated from the record of the deep extensometer to obtain a corrected value for compaction of materials within the depth interval of +88 ft to −175 ft (+27 to −53 m) relative to sea level. A graphic technique described by Riley (1969 and oral commun., 1976) indicates an elastic compaction of 0.022 ft (6.7×10^{-3} m) when the potentiometric surface declined 50 ft (15 m).

The average storage coefficient of the sediments between the exten-

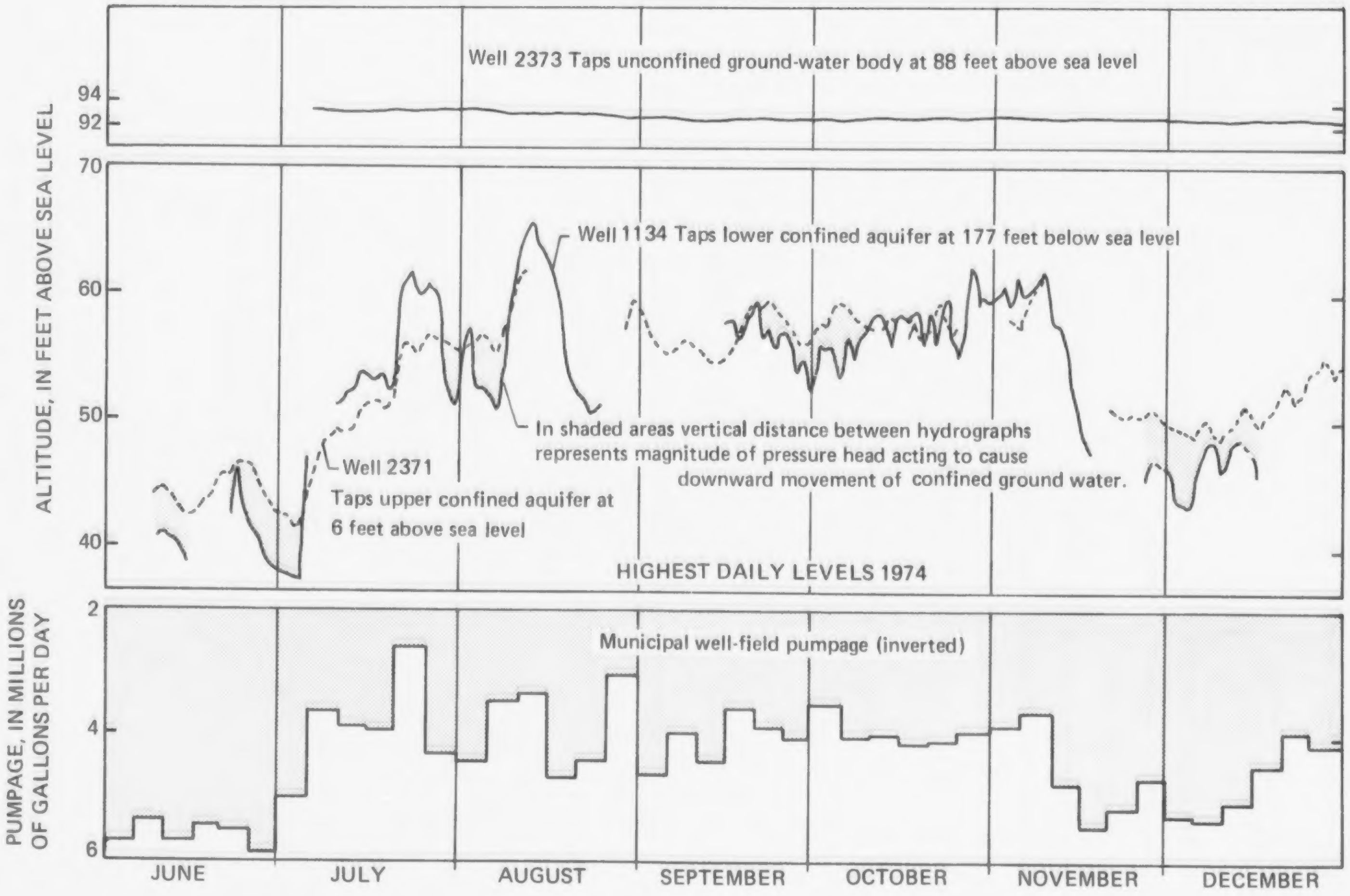


Figure 3.--Hydrographs of selected wells and histogram of pumpage from confined aquifers in the municipal well field, 1974.

someters may be determined from an equation given by Lohman (1972):

$$\Delta m = \gamma_w \Delta h \left(\frac{\overline{S}}{\gamma_w} - \frac{\theta m}{E_w} \right) \tag{1}$$

where Δm is measured compaction

γ_w is specific weight of water

Δh is change in head

θ is average porosity (assumed 25 percent)

m is total thickness of sediments

E_w is bulk modulus of elasticity of water

$\overline{S}$ is the average storage coefficient of the units.

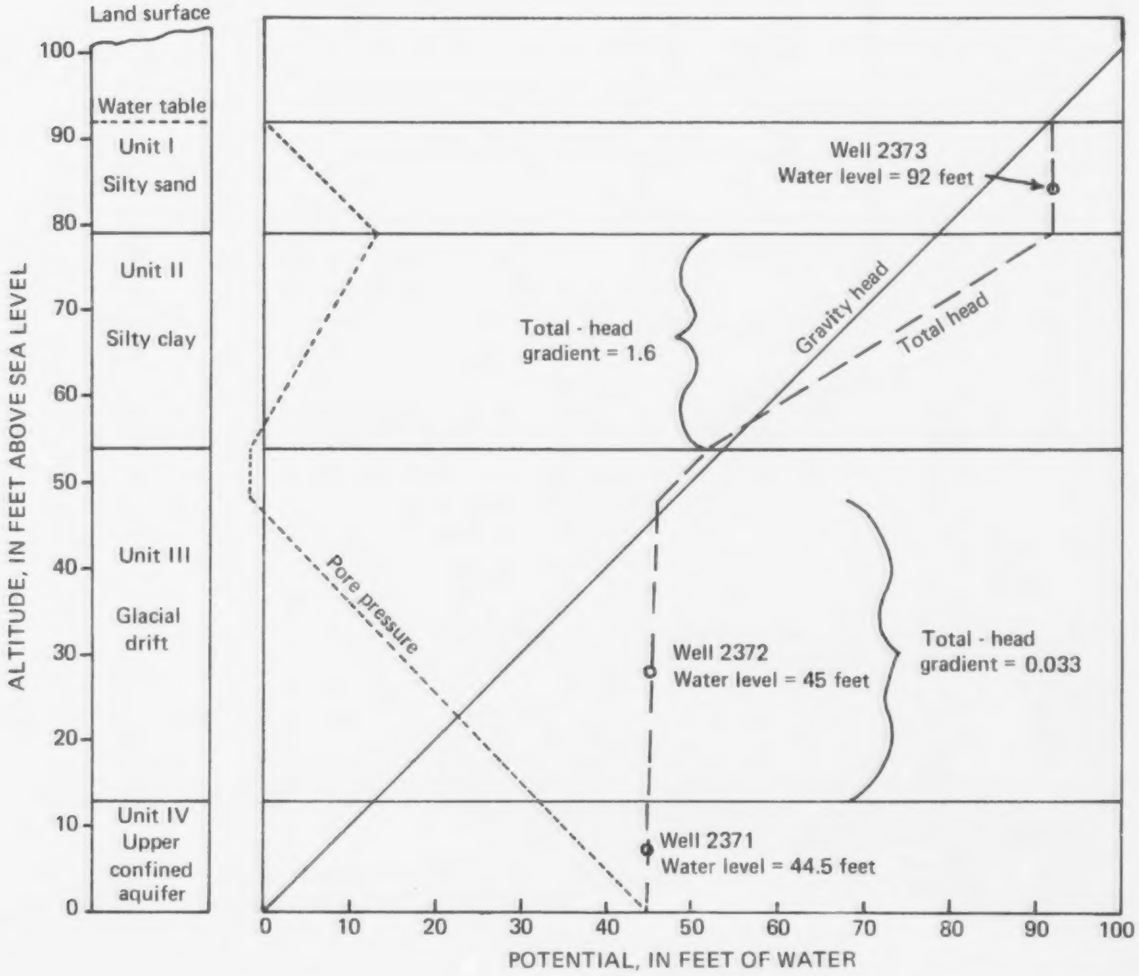


Figure 4.--Total-head gradients across Units I-IV in July 1974.

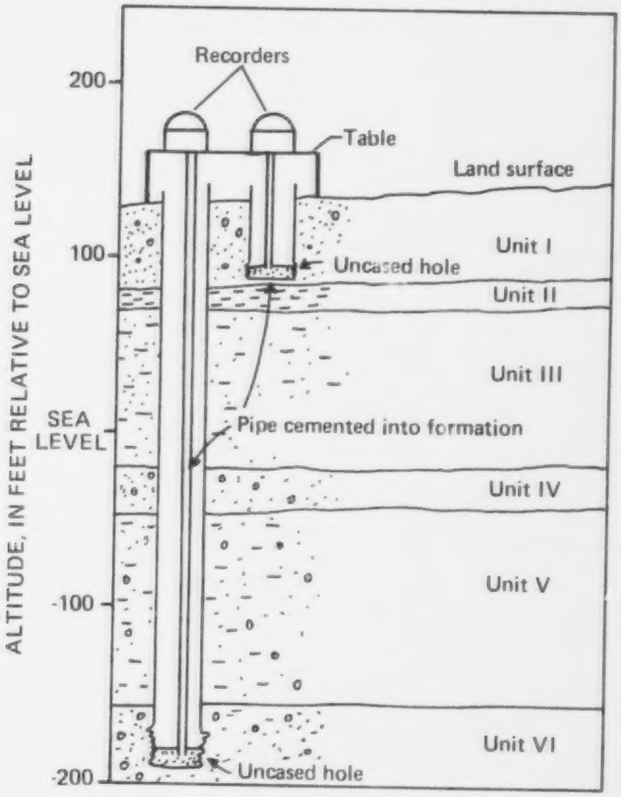

Recorders
Table
Land surface
Unit I
Uncased hole
Unit II
Unit III
Pipe cemented into formation
Unit IV
Unit V
Unit VI
Uncased hole
200
100
SEA LEVEL
-100
-200
ALTITUDE, IN FEET RELATIVE TO SEA LEVEL

Figure 5.--Extensometer installation at well 1134. Vertical movement of pipes is recorded to indicate the changes in thickness between land surface and bottom of the holes.

The total thickness of sediments refers to the interval in which a pressure reduction occurs. The extensometers measure compaction of the interval from +88 ft (27 m) to -175 ft (-53 m), or about 263 ft (80 m). However, as indicated in figure 4, the base of Unit II is under tension. If sediments in the zone of tension are unsaturated, then these sediments separate Unit II from the effects of pressure reduction in the confined aquifer. Similarly, any sediments in Unit III that are unsaturated may not be considered in the compaction calculations. For these calculations, the phreatic surface in Unit III is assumed to be at the top of the unit, at 54 ft (16.5 m) above sea level. The total thickness, m, is then +54 ft (16.5 m) to -175 ft (-53.3 m), or 229 ft (69.8 m). No correction is made for the potentially dewatered part of Unit III, because the altitude of the phreatic surface was not accurately defined.

The pressure reduction in the upper confined aquifer is assumed equal to the pressure reduction in the entire 229-foot interval. The change in head in the upper confined aquifer is 50 ft (15 m). The average storage coefficient ($\bar{S}$) of Units III, IV, and V can be computed using equation 1:

$$0.022 \text{ ft} = (62.4)(50)\left[\frac{\bar{S}}{62.4} - (0.25)(229)(2.3 \times 10^{-8})\right].$$

Solving for $\bar{S}$ yields

$$\bar{S} = 5.22 \times 10^{-4}.$$

The specific storage of the sediments is: $\bar{S}_s = \bar{S}/m$.

Then $\bar{S}_s = 5.22 \times 10^{-4}/229 = 2.3 \times 10^{-6} \text{ ft}^{-1}$
($\bar{S}_s = 7.5 \times 10^{-6} \text{ m}^{-1}$).

If the assumption that Unit II does not contribute to the compaction calculations is wrong, equation 1 must be applied to the entire 263-foot (80 m) interval spanned by the extensometer. Using a similar solution of equation 1, an $\bar{S}_s$ of 2.03×10^{-6} ft $^{-1}$(6.66×10^{-6} m^{-1}) was calculated. This value is not significantly different from the initial estimate.

Aquifer Tests

Two aquifer tests provided information on the hydrologic properties of the aquifers and confining layers. Data from both tests were analyzed using the aquitard-storage method of Hantush (1964). Hantush's equation that describes the aquifer response to constant pumping is:

$$s = \frac{Q}{4\pi T} W(u, \beta)$$

where: s is drawdown in observation well
Q is pumping rate
T is transmissivity of the aquifer

$$W(u,\beta) = \int_{u}^{\infty} 1 \; \frac{e^{-y}}{y} \; \mathrm{erfc} \left[\frac{\beta / \sqrt{u}}{\sqrt{y(y-u)}} \right] dy$$

erfc is conjugate error function

$$u \text{ is } \frac{r^2 S}{4Tt}$$

$$\beta = \frac{r}{4b} \left(\sqrt{\frac{K'S'_s}{KS_s}} + \sqrt{\frac{K''S''_s}{KS_s}} \right) \qquad (2)$$

In equation 2

r is radius to observation well
S is storage coefficient of the aquifer
t is time since pumping began
b is aquifer thickness
K is the hydraulic conductivity of the aquifer
$\left.\begin{matrix} K' \\ K'' \end{matrix}\right\}$ are vertical hydraulic conductivities of confining layers above and below, respectively, the tested aquifer
S_s is specific storage of aquifer
$\left.\begin{matrix} S'_s \\ S''_s \end{matrix}\right\}$ are specific storages of confining units above and below, respectively, the tested aquifer.

Assumptions in Hantush's analysis are:

1. Aquifer is homogenous and isotropic.
2. Flow in the aquifer is radial toward wells.
3. Flow in confining layers is vertical.
4. Drawdown effects do not fully penetrate the confining layer.
5. Well discharge is constant.
6. The aquifer responds to pumping as if the wells were fully penetrating.

The grain size of the clay (Unit VII) underlying the lower confined aquifer is much finer than that of the overlying layer (Unit V). Thus, the assumption was made that K'' is much less than K' and that S'_s and S''_s are not greatly different. Then it follows that $K'S'_s \gg K''S''_s$. For the test of the lower confined aquifer, equation 2 becomes:

$$\beta = \frac{r}{4b}\sqrt{\frac{K'S'_s}{KS_s}} \qquad (2a)$$

If the condition $K'S'_s \gg K''S''_s$ is not met, the error introduced is not great. Even if $K''S''_s = K'S'_s$, the error in $K'S'_s$ is only a factor of four. $K'S'_s$ will be greatly in error only if $K'S'_s < K''S''_s$, a condition that can be true only if the product of hydraulic conductivity and specific storage of the clay exceeds that of the gravelly sand and silt of Unit V. That possibility is considered untenable.

Figure 6 is a graph of the drawdown in wells 28 and 1134 produced by pumping well 163. The graph is a composite of data for two separate periods of pumping, one in 1956 and one in 1974. The test indicates that transmissivity is 2,650 ft^2/d (246 m^2/d) and storage coefficient is 2.4×10^{-5}. Both pairs of values for β and r should yield identical solutions for K' from equation 2a. In order for this to be true, the condition $\beta_1/\beta_2 = r_1/r_2$ must be met. However, in this test, the ratio of β's is not equal to the ratio of radii, and there is not a unique solution. Two possible values of K' can be calculated from equation 2a by substituting the possible β and r values and the values:

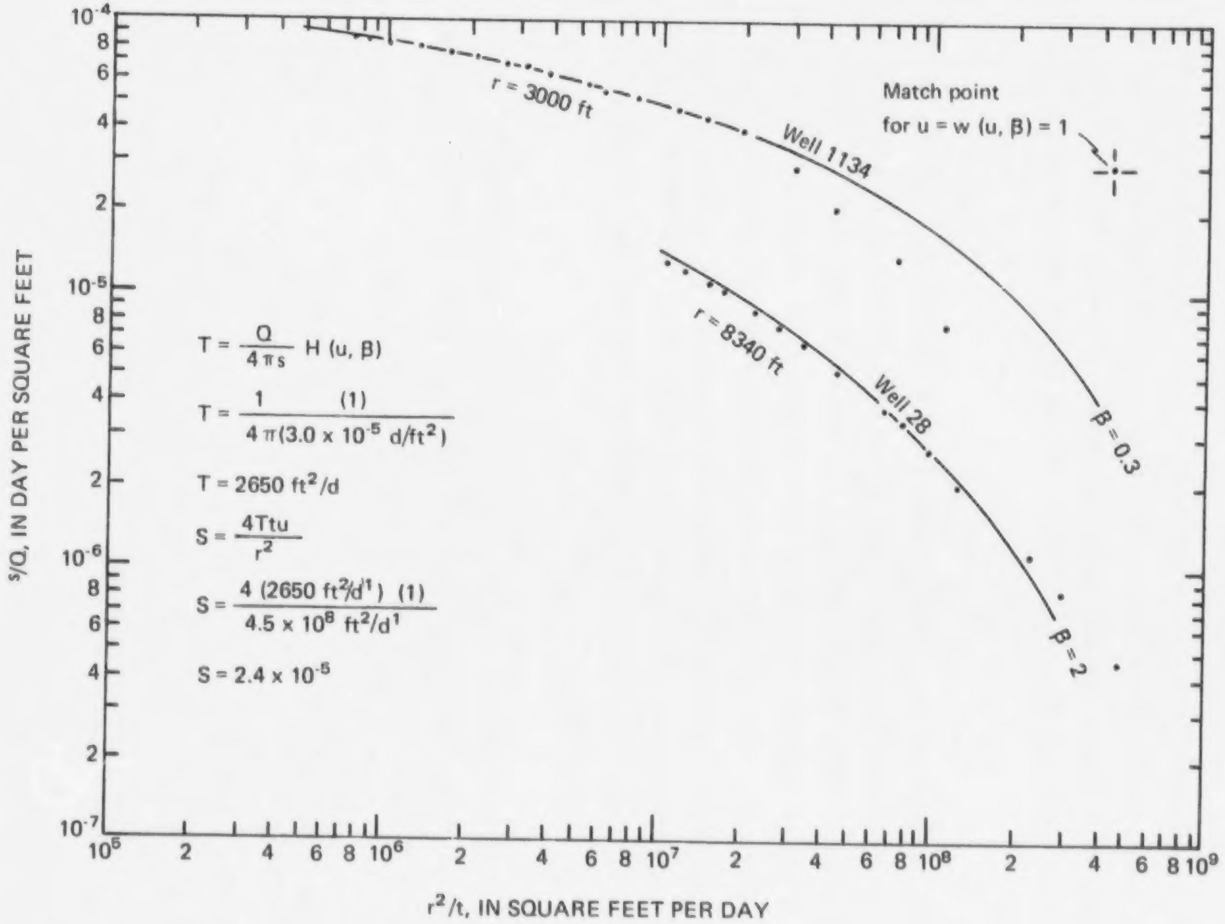
$S'_s = 2.3 \times 10^{-6}$ from the extensometer analysis
$K = T/b = 2{,}650/30$ ft/d
$b = 30$ ft
$S_s = (2.4 \times 10^{-5}/30)\ \text{ft}^{-1}$.

For $\beta = 0.3$ and r = 3,000, equation 2a yields

$K' = 4.4 \times 10^{-3}$ ft/d
($K' = 1.34$ m/d).

Similarly, for $\beta = 2$ and r = 8,340

$K' = 2.5 \times 10^{-2}$ ft/d
($K' = 7.6 \times 10^{-3}$ m/d).



$$T = \frac{Q}{4\pi s}\ H(u, \beta)$$

$$T = \frac{1 \qquad (1)}{4\pi (3.0 \times 10^{-5}\ d/ft^2)}$$

$$T = 2650\ ft^2/d$$

$$S = \frac{4Ttu}{r^2}$$

$$S = \frac{4\ (2650\ ft^2/d^1)\ (1)}{4.5 \times 10^8\ ft^2/d^1}$$

$$S = 2.4 \times 10^{-5}$$

Figure 6.--Composite graph and calculations for tests of lower confined aquifer. Water-level data are from observation wells; well 163 was pumped .

Either of these K' values is possible, but neither can be confirmed as a true solution.

In 1965, an aquifer test was performed on well 163 before it was deepened to the lower confined aquifer. Although the observation well for the test was monitored with a poorly operating pressure gage and air line, the water-level recovery in the pumped well was measured accurately using an electric tape.

Papadopulos and Cooper (1967) described a method for analyzing the drawdown in a pumped well in which casing storage is a significant part of the early pumpage. This method was used to determine the transmissivity of the upper confined aquifer based on the recovery of water levels in the pumped well (fig. 7). The transmissivity obtained by the analysis shown in figure 7, 1,320 ft^2/d (123 m^2/d), was used as a constraint in matching the questionable drawdown data from observation well 171. The match shown in figure 8 is the best fit for the data indicated and for the condition that $T = 1,320\ ft^2/d$ (123 m^2/d).

Units III and V appear to be compositionally very similar. If the specific-storage values and the hydraulic conductivities of Units III and V are assumed to be identical, then equation 2 becomes:

$$\beta = \frac{r2}{4b}\sqrt{\frac{K'S'_s}{KS_s}} \qquad (3)$$

If the assumption that $K''S''_s = K'S'_s$ is not true, the error can be no greater than a factor of four for the larger of $K''S''_s$ or $K'S'_s$. However, the smaller of the values can be anything between the larger and zero.

Solving equation 3 for the conditions

$r = 2,450$ ft

$b = 14$ ft

$K = T/b = 1,320/14$ ft/d

$S_s = S/b = 1.1 \times 10^{-5}/14\ ft^{-1}$

$\beta = 1.5\ ft^{-1}$

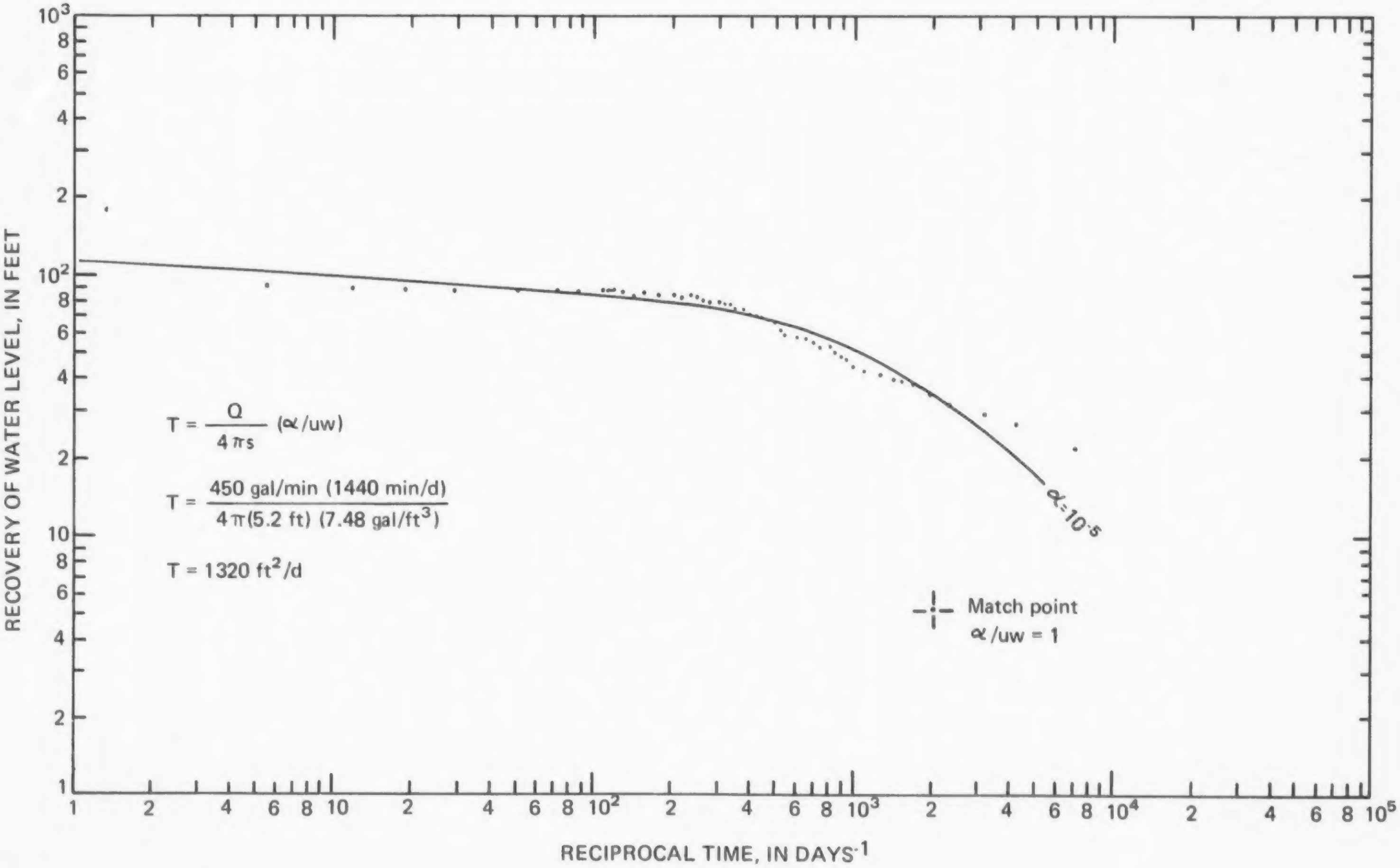


Figure 7.--Graph and calculations for recovery phase of test of upper confined aquifer. Data are from pumped well 163.

$S'_s = 2.3 \times 10^{-6}\ ft^{-1}$ (from extensometer analysis)

yields $K' = 9.5 \times 10^{-3}$ ft/d ≈ 0.01 ft/d
($K' = 2.9 \times 10^{-3}$ m/d).

The first aquifer test gave values of the vertical hydraulic conductivity of 4.4×10^{-3} ft/d (1.34×10^{-3} m/d) and 2.5×10^{-2} ft/d (7.6×10^{-3} m/d). The second test indicates a value of about 0.01 ft/d (3×10^{-3} m/d). The average of the two values from the first test (to one significant figure) is also 0.01 ft/d (3×10^{-3} m/d). Thus the assumption that the vertical hydraulic conductivity of Unit III equals that of Unit V is not disproved. A laboratory analysis of a drive-core sample from Unit

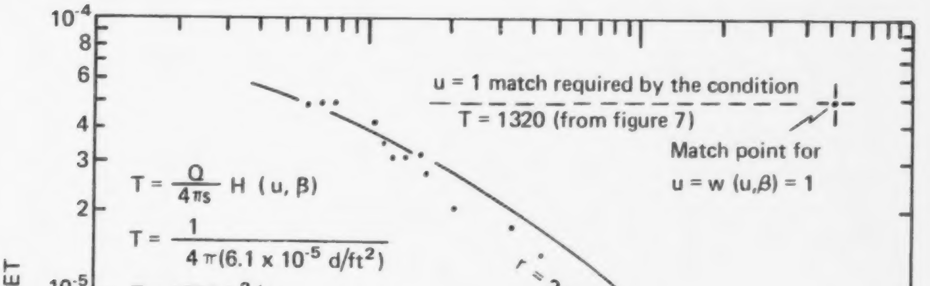


Figure 8.--Graph and calculations for drawdown phase of test of upper confined aquifer.

III (see sample D in table below) indicated a vertical hydraulic conductivity of .031 ft/d (9.5 x 10^{-3} m/d). The laboratory value differs by only a factor of three from our estimate of 0.01 ft/d (3 x 10^{-3} m/d). This is considered adequate agreement. Laboratory results commonly differ from field results because cores are disturbed during drilling and when placed in experimental devices. The value of 0.01 ft/d (3 x 10^{-3} m/d) is considered the best estimate of vertical hydraulic conductivity of Units III and V.

Geohydrologic properties of drive-core samples

	CORE			
	A	B	C	D
Well number	2372	2371	2371	2372
Depth below bottom of refuse (feet)	14	18	20	44
Altitude above sea level (feet)	76	72	70	46
Unit cored	II	II	II	III
Median grain size (millimeters)	0.031	0.015	0.024	0.050
Clay-size particles (percent 0.004 millimeters)	52	30	57	10
Moisture content (percent by volume)	37	44	41	30
Vertical hydraulic conductivity (feet/day at 4°C and unit gradient)	$1.3\text{x}10^{-4}$	$0.9\text{x}10^{-4}$	$0.1\text{x}10^{-4}$	$3.1\text{x}10^{-2}$

Vertical Seepage

The downward flux of ground water per unit area is defined by

$$Q_{III} = K'_{III} \left(\frac{dh}{dl}\right)_{III} ,$$

where Q_{III} is flux per unit area of Unit III (in ft/d)

K'_{III} is vertical hydraulic conductivity of Unit III (in ft/d)

$\left(\frac{dh}{dl}\right)_{III}$ is hydraulic gradient through Unit III, as defined in figure 4.

Then $Q_{III} = 1 \times 10^{-2} (0.033) = 3.3 \times 10^{-4}$ ft/d
($Q_{III} = 1.0 \times 10^{-4}$ m/d).

At steady-state conditions, the downward flux through Unit III is equal to the downward flux through Unit II.

Thus $$Q_{II} = Q_{III} = K'_{II} \left(\frac{dh}{dl}\right)_{II}$$

and substituting values for Q_{III} and $\left(\frac{dh}{dl}\right)_{II}$

yields $3.3 \times 10^{-4} = K'_{II}$ (1.6).

Then $K'_{II} = 2 \times 10^{-4}$ ft/d
($K'_{II} = 6 \times 10^{-5}$ m/d).

This calculated value for the vertical hydraulic conductivity of Unit II differs by a factor of less than three from the average of the laboratory values determined from cores A, B, and C (see table). This is an acceptable agreement and the author considers the field value of hydraulic conductivity, 2×10^{-4} ft/d (6×10^{-5} m/d) to be a "best estimate" for K'_{II}.

Theoretical velocity calculations, based on the equation (Lohman, 1972):

$$V_s = \frac{K}{\theta} \frac{dh}{dl}$$

are useful in providing approximate flow rates of water, which may contain pollutants, in various materials. In non-uniform sediments, small amounts of pollutants commonly travel faster than the theoretical rates by following preferred high-permeability paths. The theoretical rate of migration of the pollutants is also greatly affected by adsorption and dispersion. These factors may greatly retard the rate at which the concentrations of pollutants in the water reaching an aquifer increase to the "original" concentration of the leachate at its point of origin (base of a landfill). The dispersion and adsorption characteristics of the sediments beneath the Merrill Field landfill have not been determined.

The following calculations should be construed as giving broad ranges of travel times without regard to the lyophilic or lyophobic nature of the soil-water-pollutant solutions. They should be viewed in the context of the question, "Will breakthrough of pollutants to the upper confined aquifer be in terms of days, years, or centuries?"

The seepage velocity through Unit II is defined by:

$$V_s = \frac{K''_{II}}{\theta} \left(\frac{dh}{dl}\right)_{II}$$

where θ is porosity of Unit II.

θ is assumed to be equal to 0.35,

and $$V_s = \frac{2 \times 10^{-4}}{0.35} (1.6) = 9.1 \times 10^{-4} \text{ ft/d.}$$

Then, if a molecule of water moves at that seepage velocity through Unit II, the transit time through the 25-foot thick unit would be:

$$\text{Time} = \frac{\text{Distance}}{\text{Velocity}} = \frac{25 \text{ ft}}{9.1 \times 10^{-4} \text{ ft/d}} = 27{,}473 \text{ d, or 75 yr (rounded).}$$

The time it takes for the same molecule of water to migrate downward through Unit III to the upper confined aquifer can be calculated similarly.

Assuming the porosity of Unit III is 0.25, the seepage velocity is:

$$V_s = \frac{1 \times 10^{-2}}{0.25}\ (.033) = 1.3 \times 10^{-3}\ \text{ft/d.}$$

The transit time through Unit III, which is 42 ft (12.8 m) thick, is then:

$$\text{Time} = \frac{42\ \text{ft}}{1.3 \times 10^{-3}\ \text{ft/d}} = 32{,}308\ \text{d, or 89 yr (rounded).}$$

The total transit time through Units II and III to the upper confined aquifer is 75 years + 89 years = 164 years.

The initial breakthrough of landfill-derived pollutants to the upper confined aquifer would presumably be much earlier than this. A theoretical breakthrough curve (fig. 9) indicates that very low concentrations of pollutants may begin entering the aquifer after about 0.5 transit times (80-85 years), but that the effluent will not reach the concentration of leachate in the unconfined aquifer until after 1.5 transit times. In clay-rich soils even greater attenuation may occur.

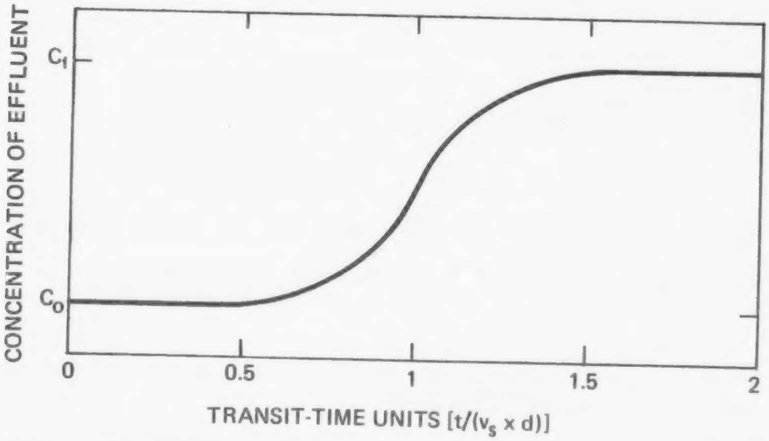


Figure 9. -- A theoretical breakthrough curve, C_0 and C_1 are concentrations of unpolluted pore water and landfill leachate, respectively. Transit-time units (time divided by theoretical transit time) are equivalent to pore-volume units of other authors.

Discussion of Test Results

From the extensometer analysis it was determined that the average specific storage of the sediments is 2.3×10^{-6} ft^{-1} (7.54×10^{-6} m^{-1}). The part of this value that results from compaction of the aquifer skeleton is (Lohman, 1972):

$$S_{as} = \frac{\gamma \Delta b}{b \Delta p}$$

where γ is specific weight of water

Δb is the measured compaction (change in thickness, b)

b is the aquifer thickness

Δp is the change of head,

provided that the aquifer skeleton behaves elastically. Then:

$$S_{as} = \frac{(62.4)\ (.022)}{(50\ \text{ft})\ (62.4)\ (229)} = 1.9 \times 10^{-6}\ \text{ft}^{-1}.$$

The specific storage due to compressibility of water is: $S_w = \theta\gamma/E_w$

where θ is porosity of sediments

E_w is the bulk modulus of elasticity of water.

Then $S_w = (0.3)\ (62.4)/4.32 \times 10^7 = 0.43 \times 10^{-6}$ ft^{-1}.

The specific storage due to elastic compaction of the aquifer is therefore 4.4 times that due to expansion of water. However, in both aquifer tests, the storage coefficients were about equally attributable to compression of the aquifer and expansion of water.

To evaluate the effect of underestimating specific storage an assumption was made that all sediments have a specific storage equal to that determined from the extensometer, 2.3×10^{-6} ft^{-1} (7.5×10^{-6} m^{-1}). If all sediments have the same specific storage, then equation 2 becomes:

$$\beta = \frac{r}{4b}\left(\sqrt{\frac{K'}{K}} + \sqrt{\frac{K''}{K}}\right).$$

Recalculating K' for the test of the upper confined aquifer and the match $\beta = 0.3$ at r = 3,000 ft (915 m), yields

$$0.3 = \frac{3000}{4\ (30)} \sqrt{\frac{K'}{2650/30}}$$

or

$$K' = .0127 \text{ ft/d}$$
$$(K' = 3.9 \times 10^{-3} \text{ m/d}).$$

This is about three times the previously determined value. Similarly for $\beta = 2$ and r = 8,340 ft (2,540 m), the value is about three times the previously determined value. For the test of the lower confined aquifer, equation 2 becomes:

$$\beta = \frac{r}{2b} \sqrt{\frac{K'}{K}}\ .$$

Then
$$1.5 = \frac{2450}{2\ (14)} \sqrt{\frac{K'}{1320/14}}$$

or
$$K' = .028 \text{ ft/d}$$
$$(K' = 8.5 \times 10^{-3} \text{ m/d}).$$

Again, this is nearly three times as great as the value of 0.01 ft/d, (3×10^{-3} m/d) determined previously from the test of the upper confined aquifer. Although calculations based on a uniform specific storage are useful in assessing possible errors in the vertical hydraulic conductivities, these new values were not used. If it were concluded that the aquifer tests gave erroneous values for the storage coefficients, then it must also be concluded that the values for β are in error. However, there is no basis for selecting alternate values for β. Furthermore, the new K' values reduce the transit-time calculation by a factor of only three. After dividing previous times by a factor of three, the time of first breakthrough remains in the order of tens of years and the full breakthrough remains in the order of hundreds of years.

Throughout this report data from aquifer tests dating back almost 30 years were used. It appears that these data can not be used to analyze vertical hydraulic conductivities accurately. The maximum field value for hydraulic conductivity of Units III and V was 2.5×10^{-2} ft/d (7.62×10^{-3} m/d), which is more than twice as great as the author's best estimate. Similarly, the minimum field value was 4.4×10^{-3} ft/d (1.34×10^{-3} m/d), which is less than half of my best estimate. Carrying these maximum and minimum values through the calculations suggests transit times from as little as 65 years to as great as 368 years. Thus, the margin of error is likely to be a factor of two to three. If more accurate values are needed, new experiments must be designed.

Only two aspects of vertical migration of pollutants were considered--vertical gradients and vertical hydraulic conductivities. The author has no data on dispersion and adsorption characteristics of the aquifers and confining beds. These characteristics must also be determined before migration of pollutants can be analyzed fully.

It is also recognized that there is flow toward the production wells from areas not overlain by the landfill. This flow would dilute any leachate seeping through the confining layers to the aquifers. Again, a more detailed analysis would require that new experiments be designed and new models be developed.

Conclusions

1. Vertical gradients under the Merrill Field landfill vary with changes in pumping. However, using the conditions of July 1974 as typical of normal pumping, the gradients through the confining Units II and III are 1.6 and 0.033, respectively.

2. The "best estimate" of the vertical hydraulic conductivity of the gravelly sand and silt of Units III and V is 1×10^{-2} ft/d (3×10^{-3} m/d).

3. The "best estimate" of the vertical hydraulic conductivity of Unit

II is 2 x 10^{-4} ft/d (6 x 10^{-5} m/d).

4. The data do not support a unique and accurate value of vertical hydraulic conductivity for any of the units. Values may be in error by plus or minus a factor of two or three.

5. Theoretical seepage calculations indicate that minor amounts of pollutants may reach the upper confined aquifer after many tens of years, but that water of the composition of the leachate may not reach the upper confined aquifer for more than two centuries.

6. Although there have been a number of aquifer tests performed since the 1950's, they have not been designed to obtain the type(s) of data necessary to predict migration of pollutants accurately. If such predictions are needed, new experiements and models must be designed.

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Session VII
Containment/Control Systems for Landfills and Impoundments

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A NEW COMPOSITE LINER FOR HAZARDOUS WASTE IMPOUNDMENTS

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Abstract

A composite liner made of a synthetic membrane placed between two 6-in.-(152-mm) compacted lifts of soil-cement has been developed by the Portland Cement Association. Construction feasibility was demonstrated by a test section built in September 1983 near Apalachin, N.Y. Observations show no damage occurred to the membrane. The soil-cement serves as a foundation and protective cover for the membrane. This composite liner is strong, durable, and impermeable with an estimated in-place construction cost of $1.30/sq ft ($14/sq m). Conventional highway construction equipment is used.

For over 35 years soil-cement has successfully demonstrated its ability to serve as a low permeable liner for various types of impoundments. More recently laboratory testing of soil-cement exposed to certain hazardous wastes has shown it to be technically feasible as a lining material.

Soil-cement is durable having been exposed to wave action, freezing and thawing cycles, wetting and drying cycles, sea water, municipal and industrial waste-water, brine solutions, coal ash slurries, longitudinal flows, and sanitary landfill leachates.

Soil-cement is strong, which makes it a good working surface for mechanical equipment. This enables use of frontend loaders, tractors, or other equipment to deposit or remove waste materials.

It is environmentally acceptable because usually the excavation from which the soil is obtained becomes part of the lagoon or pond and, thereby, covered with, for example, wastewater. Construction does not leave "scars" on the environment.

The bottom layer of soil-cement is mixed in place. The top layer is premixed and placed on the membrane and spread by a dozer. Static and vibratory rollers are used to compact the soil-cement to the proper density. The membrane thickness can be a minimum of 30 mil.

Soil-Cement As a Material

Soil-cement can be broadly defined as a mixture of natural soils, portland cement, and water. In its most-compacted state it forms a strong, durable, economical, and virtually impermeable material with properties similar to concrete and natural rock. American Society for Testing and Materials standard tests in connection with other criteria are used to determine, for any given soil, the cement content and moisture content necessary to produce the strength, durability, and other characteristics that may be desired. A typical mixture contains about 10% cement by weight of the dry soil but actual cement contents have ranged from less than 7% to more than 14% by weight.

Advantages of Soil-Cement

The durability of soil-cement has been most significantly demonstrated by the satisfactory performance of a 33-year-old test section built by the U.S. Bureau of Reclamation. Located on the southeast shore of the Bonny Reservoir in Eastern Colorado, this test section has been subjected to severe wave action producing countless cycles of wetting and drying and more than 100 cycles of freezing and thawing each year. The soil-cement test section is 24 ft (7 m) high and about 300 ft (91 m) long. It consists of a series of 6-in. (152-mm)- thick by 7-ft (2-m)-wide horizontal layers of soil-cement, one on top of another. Each successive layer is offset 12 in. (305 mm) from the preceding layer so that the exposed, finished embankment slope is 2:1 (horizontal to vertical) (Fig. 1). (Coffey, C. T. and Jones, C. W., 1961).

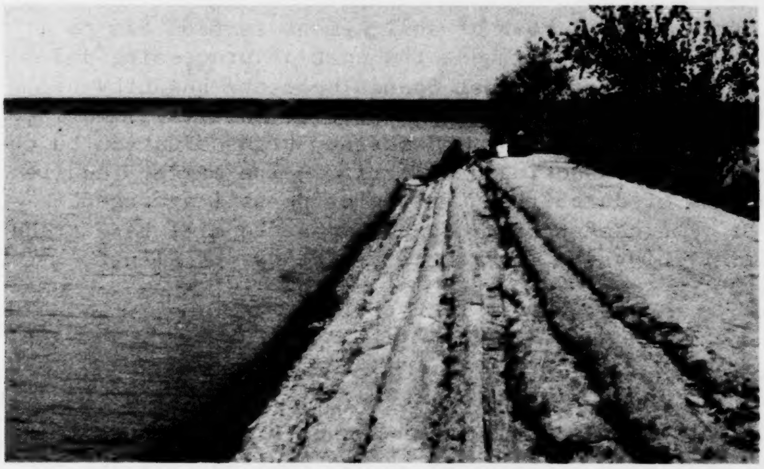

Figure 1. Soil-Cement Test Section, Bonny Reservoir, Colorado, after 29 years of exposure.

In addition, there have been many other projects that have satisfactorily withstood these exposure conditions. Soil-cement durability also has been demonstrated by projects exposed to seawater, domestic and industrial wastewater, longitudinal flows, coal ash slurries, and brine solutions. Laboratory tests sponsored by the U.S. Environmental Protection Agency show soil-cement is compatible with some hazardous wastes and sanitary landfill leachates. (U.S. Environmental Protection Agency, 1983).

Soil-cement continues to gain strength with age. Seven-day compressive strengths are from 500 psi (3,448 kN/m^2) to 1000 psi (6,895 kN/m^2) to 1500 psi (10,343 kN/m^2). Seventeen-year-old cores taken from the soil-cement slope protection serving Merritt Dam in Nebraska exhibited very high strength gains. The average compressive strength was 3754 psi (25,884 kN/m^2) with one core having a compressive strength of 5165 psi (35,613 kN/m^2). The cement content used for construction of this project was 14% by volume. This strength is important to ensure a firm working surface when soil-cement is used to line the bottom of an impoundment. A firm working surface enables use of front-end loaders, tractors, and clam shells to remove or place solids in these basins. Soil-cement can provide sufficient structural integrity for impoundment dikes and serve as an adequate foundation underneath a synthetic liner.

Soil-cement is virtually impermeable with permeability coefficients as low as 0.10 ft/yr or 10^{-7} cm/sec. (Portland Cement Association, 1975). Shrinkage cracks develop in soil-cement but the majority of crack openings are less than 1/16 in. (1.6 mm) wide and spaced for granular soils somewhere between 10 to 20 ft (3 to 6 m). (Highway research Board, 1961). Laboratory tests show that based on a conservative assumption of crack size and spacing, seepage increased only 5%. (Nussbaum, P. J. and Colley, B. E., 1971). From a practical standpoint this is insignificant considering the low permeability of soil-cement. In addition, field observations show these cracks tend to be filled with the sediment that settles on the bottom of basins.

In general the current total cost of soil-cement is from $25 to $30/cu yd ($33 to $39/$m^3$). This includes the cost of processing and the cost of cement. The processing cost dpends upon the quantity of soil-cement to be placed. This includes the cost of material, hauling, central plant mixing, water for mixing and curing, transportation to the placement area, spreading, compacting, and curing. The cement required depends upon the results of durability testing using soil aggregate proposed for the job. The cement cost depends upon the mill base price and the cost of delivery plus the contractor's cost for handling, overhead, and profit.

Laboratory Tests for Soil-Cement

Laboratory tests for the required cement and moisture factors in soil-cement projects have been established by the American Society for Testing and Materials (ASTM). Optimum moisture and maximum density are determined by ASTM test D-558. The required cement content is determined by ASTM D-559 and D-560 plus the Portland Cement Association weight-loss criteria. (Portland Cement Association, 1976).

Soil-Cement As A Liner

Soil-cement has had a long and successful history as a liner for all types of storage basins. In 1945, soil-cement was first used to line the bottom of a 12-million-gallon (45,420 m^3) water storage reservoir located in Port Isabel, Texas. The reservoir had been constructed initially with concrete-paved side slopes and no lining on the bottom. Leakage was so extensive, however, that steel petroleum storage tanks nearby were floated. With soil-cement placed on the bottom there was no leakage at the end of two years. After 23 years of service the owner reported that the soil-cement was still performing satisfactorily and that maintenance was negligible. (Rogers, E. H., 1968). Presently, there are not any problems with this liner.

Through 1982, about 250 energy and water resource-related soil-cement projects have been constructed in the United States and Canada. (Portland Cement Association, 1981). These projects range in size from a few thousand cubic yards to over one million cubic yards of soil-cement for the slope protection serving the cooling water reservoirs at the South Texas Power Plant near Bay City, Texas. Since 1963 soil-cement has been used in 68 recreational development lake, water storage reservoir, wastewater treatment lagoon, and cooling water reservoir projects. (Fig. 2 and 3).

Soil-Cement/Synthetic Membrane Lines

The use of soil-cement and concrete in conjunction with synthetic material for lining basins is not new. About 1954 the Colorado River Water District, Big Springs, Texas, constructed an 80-million-gallon

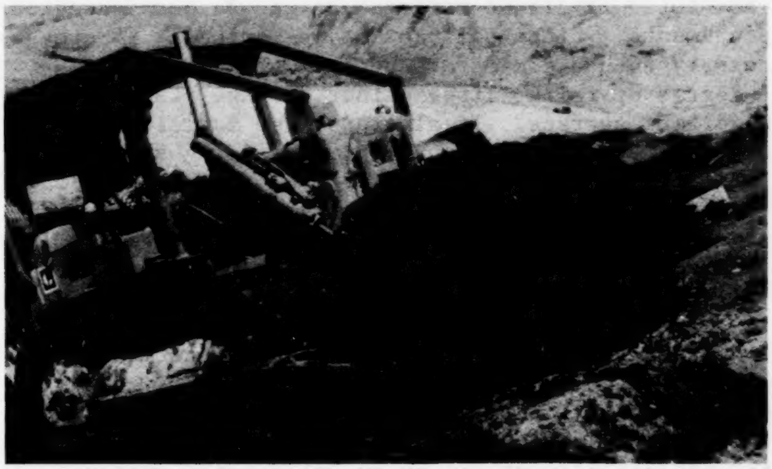

Figure 2. Spreading Soil-Cement on Side Slope, Riverside Wastewater Lagoons, California.

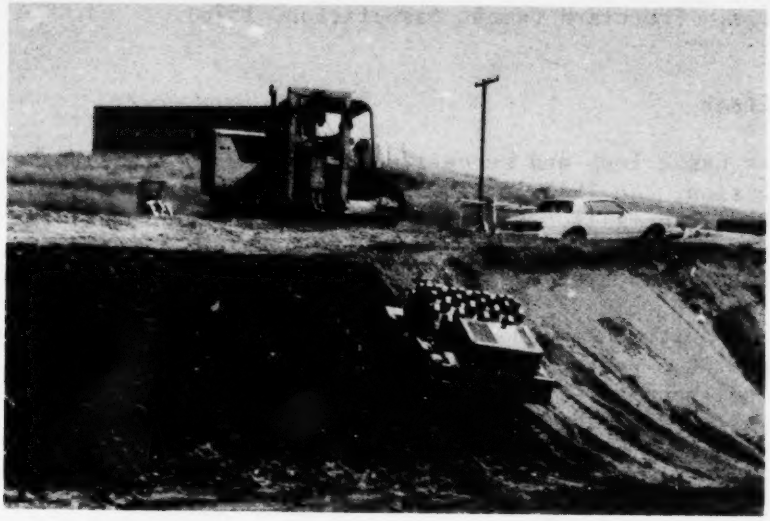

Figure 3. Initial Compaction of 8-in. (203-mm)-thick Soil-Cement, Riverside Wastewater Lagoons, California.

water storage reservoir in Odessa, Texas. The side slopes serving this reservoir are 3 to 1 (horizontal to vertical). To prevent seepage the bottom and side slopes were lined with 4 in. (102mm) of soil-cement placed over prefabricated asphalt panels. No seepage problems have been reported. Other composite liners using gunite and concrete placed over asphalt panels and plastic membranes have successfully been used in water storage reservoirs and irrigation canals during the past 29 years.

A composite liner consisting of a synthetic membrane placed between two 6-in. (152-mm) compacted layers of soil-cement can be considered at hazardous waste impoundments. Since soil-cement is virtually impermeable it would not be necessary to use a membrane thicker than 30 mil. The liner offers a high degree of reliability against seepage, is durable, and provides a firm working surface. It also provides additional impoundment storage volume over conventional thicker lining systems currently in use. The virtual impermeability of soil-cement should reduce the concentration of hazardous wastes contacting the synthetic membrane. This would help increase the longevity of the membrane. This composite liner is economical and compares favorably with competitive lining systems in use today.

The composite liner offers the following additional benefits:

1. It resists degradation. Laboratory tests sponsored by the Environmental Protection Agency showed that no seepage had occurred through soil-cement following 2-1/2 years exposure to certain hazardous wastes. After 625 days of exposure to these wastes the compressive strength of the soil-cement exceeded the

compressive strength of similar soil-cement that had not been exposed to the wastes. These wastes were toxic pesticide formulations, oil refinery sludge, toxic pharmaceutical wastes, rubber, and plastic wastes. Soil-cement was rated "fair" in containing caustic petroleum sludges, indicating that the specific combination of soil-cement and waste should be tested. Other wastes being considered for containment should also be tested for compatibility with soil-cement. The high density polyethylene material resists degradation to a broad range of wastes.

2. This liner will be of sufficient strength and thickness to prevent failure due to physical stress (e.g., earth-moving equipment and the weight of large volumes of wastes). Since soil-cement does continue to gain strength with age front-end loaders, clam shells, and farm-type tractors can be and have been successfully used for solids handling at various impoundments. The soil-cement can protect the synthetic membrane from damage.

3. The foundation underneath the synthetic membrane is capable of providing adequate support and resisting pressure gradients. Soil-cement has been used as a massive fill to provide foundation strength and uniform support under large principal structures. In Koeberg, South Africa, for example, soil-cement was used to replace an approximate 18-ft (5.5-m)-thick layer of medium dense saturated sand under two 900-MW nuclear power plants. Although the in-situ sand was capable of carrying the static loads, preliminary studies showed that the factor of safety against liquefaction was less than 1.0; therefore, some foundation treatment was required to satisfy the dynamic loading. An extensive laboratory testing program was conducted to determine static and dynamic design characteristics, liquefaction potential, and durability of the soil-cement. Results showed that with only 5% cement content by dry weight, cohesion increased significantly and it was possible to obtain a material with strong enough chemical bonds between the sand grains to prevent liquefaction.

4. This composite liner can be placed to cover all surrounding earth likely to be in contact with the waste or leachate. Thus, lateral as well as vertical migration will be prevented.

Construction of the Composite Liner

The composite liner can be constructed with conventional paving equipment. This was demonstrated on September 20-23, 1983 when a test section was successfully constructed near Apalachin, NY. It simulates a portion of a waste impoundment and has a horizontal length of 30 ft (9 m) and a side slope length of 40 ft (12 m). The side slope is 3 to 1 (horizontal to vertical). The test section is 44 ft (13.4 m) wide. Both a 30-mil and a 40-mil high density polyethylene membrane were

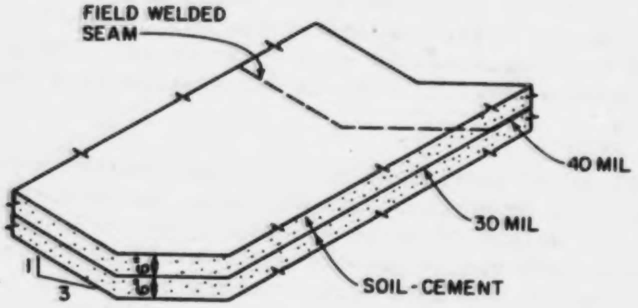

Figure 4. A Section Drawing of the Composite Liner Test Project, Apalachin, N.Y.

placed adjacent to each other on the bottom 6-in. (152-mm)-compacted layer of soil-cement. The membranes were field welded (Fig. 4). Observations of each membrane immediately following compaction of the top layer of soil-cement showed that they are puncture-resistant to the placement of soil-cement even with 3/4-in. (19-mm) aggregate placed under them. There was also no damage to the seam. Solid Seal Systems, Inc., Apalachin, New York, was the soil-cement contractor. Gundle Lining Systems, Inc., Houston, Texas, provided the synthetic membrane. Observations in July, 1984 show no damage to the soil-cement and the synthetic membrane due to alternate cycles of freezing and thawing and wetting and drying this past winter and spring.

The 6-in. (152-mm) compacted layer of soil-cement located below the membrane was constructed with the mixed-in-place method. This method involves proportioning cement on a loose thickness of soil sufficient for the design compacted thickness. The cement is mixed throughout this thickness. Water is added in sufficient quantity to result in a final soil-cement mixture at optimum moisture. Mixing of soil-cement and water is done followed by compaction. A smooth steel wheel roller was used for compaction. The soil used on this bottom layer had a A-1-b classification with 100 percent passing the 2-in. (51-mm) sieve. Approximately 77 percent and 3 percent passed the No. 4 sieve and No. 200 sieve, respectively. The cement content was 5 percent and the moisture content was about 7 percent. The bottom layer of soil-cement was cured with water. The 7-day compressive strength was 541 lbs/in^2 (3,730 kN/m^2).

The 22-1/2-ft (6.9-m)-wide 40-mil membrane panel was placed on the smooth soil-cement surface by rolling it down along one edge of the side slope and on to the horizontal portion of the test section. One edge of the membrane was placed in a trench that had been excavated at the crest of the side slope. The membrane was anchored on the test section by backfilling that portion of the trench. The 22-1/2-ft (6.9-m)-wide 30-mil membrane panel was placed in a similar manner adjacent to the 40-mil panel. The panels were field welded.

Soil-cement for the top layer was mixed in a flat area adjacent to the test section. It can also be processed in a central mixing plant. The same soil used for the bottom layer of soil-cement was used for the top layer. The cement content was 7 percent and the moisture content was 8.8 percent. Initial placement of the premixed soil-cement was at the leading edge of the horizontal portion of the 40-mil membrane panel. It was dumped at this location and spread with a dozer to a loose thickness sufficient to attain a compacted thickness of 6 in. (152 mm). Placement continued along a majority of the horizontal section before placement on the side slope. The soil-cement was dumped at the toe of the side slope and spread on the slope (Fig. 5). Construction progressed until about 34 ft (10.4 m) of the top layer width of soil-cement had been placed on the horizontal area and side slope. This left approximately 10 ft (3 m) of the 30-mil membrane uncovered.

A smooth steel wheel roller was used for compacting this top layer of soil-cement (Fig. 6). One pass without vibration and one pass with vibration were made. The density achieved was 92 percent and 90 percent of maximum density for the horizontal portion and side slope, respectively. This was satisfactory for purposes of the test section but it is recommended at least 96 percent of maximum density be

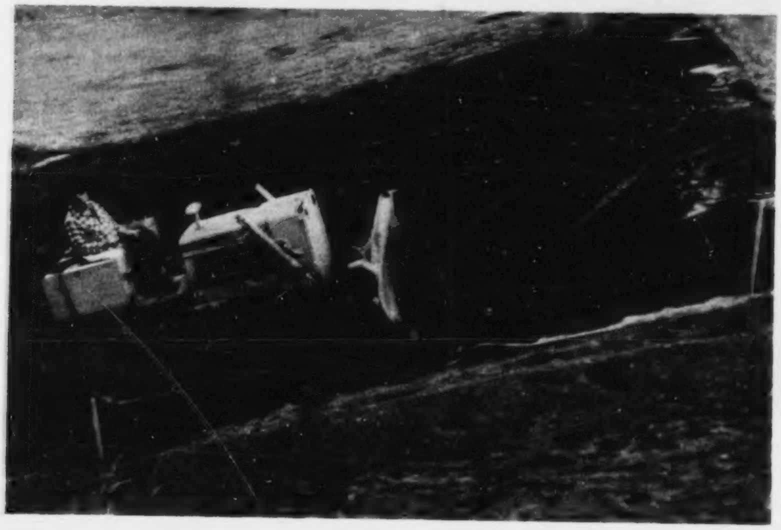

Figure 5. Spreading Soil-Cement on Membrane at Side Slope, Apalachin, N.Y.

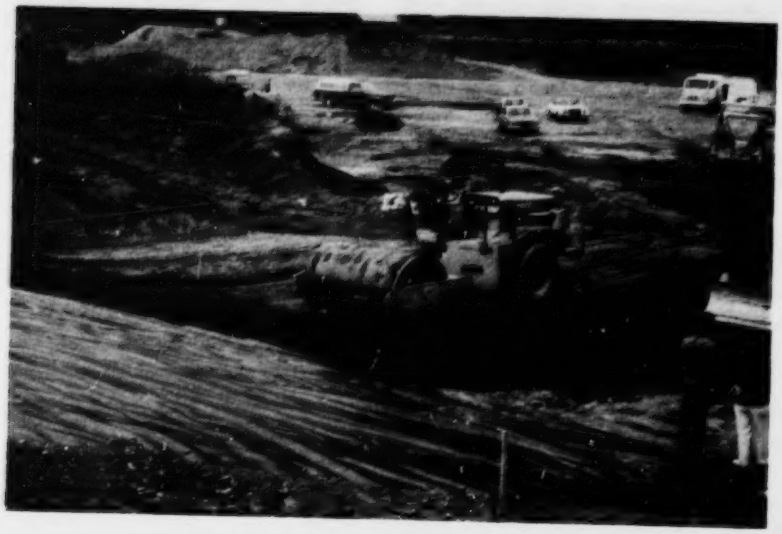

Figure 6. Compacting Soil-Cement on Membrane at Horizontal Area, Apalachin, N.Y.

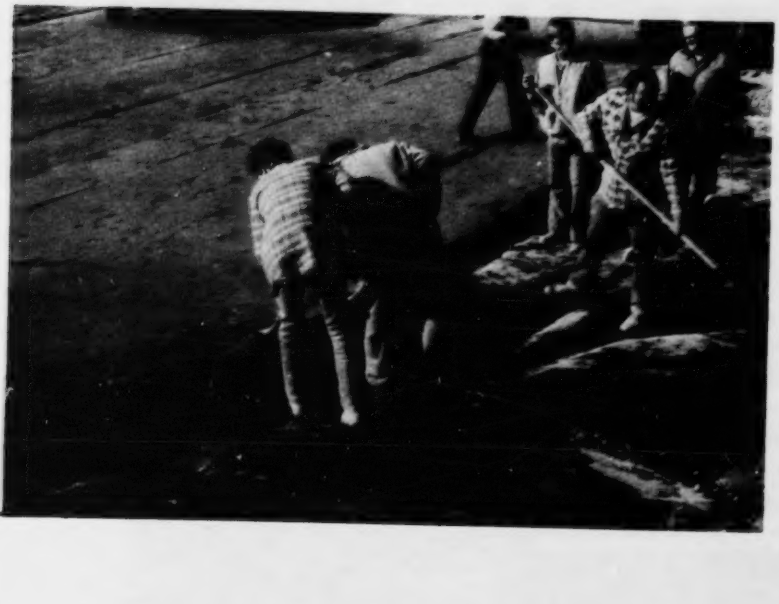

Figure 7. Making a Construction Joint in Top Layer of Soil-Cement, Apalachin, N.Y.

achieved. The compacted soil-cement was cured with water. The 7-day compressive strength ranged from 851 lbs/in.^{2} (5,872 kN/m^2) to 907 lbs/in.^{2} (6,254 kN/m^2). The purpose in temporarily terminating placement of soil-cement as reported above was to determine the feasibility of making a construction joint in the soil-cement without damaging the 30-mil membrane. Two types of joints were successfully made with a shovel (Fig. 7). One was at 45 degrees and the other a vertical joint. We currently recommend use of vertical construction joints.

Hazardous Waste Impoundment Dikes

Waste impoundment must have dikes that are designed, constructed, and maintained with sufficient structural integrity to prevent massive failure. At some locations the only readily available material for embankment construction is very fine beach sand, which is unstable against wind, rain, and wave action. However, excellent soil-cement can be made from this material and embankments constructed completely of such soil-cement are resistant to these exposure conditions. This type of solid soil-cement embankment was used to form the 1100-acre (4.45×10^6 m^2) cooling water reservoir for the Barney M. Davis Power Plant near Corpus Christi, Texas. The reservoir consists of 6.5 miles (10.5 Km) of circumferential embankment and 2.1 miles (3.4 Km) of interior baffle dikes, all constructed of soil-cement. This construction material not only protects the interior slopes of the reservoir from wave action created by the stored water but the exterior slopes are also protected from waves generated by exterior flooding and heavy rains caused by hurricanes. In addition to the unique embankment section, this project used salt water in the soil-cement mixing process. Laboratory tests indicated that the salt water produced no apparent harmful effects in the completed mixture and even increased the 7- and 28-day compressive strengths. The cooling water handled in the reservoir is also salt water. The construction of the embankment and dikes as well as filling the reservoir were completed in 1973. To date there has been no indication of seepage or detrimental erosion of the solid soil-cement embankments.

Impoundment Cover

This composite liner section is a feasible alternative as a final cover serving a waste impoundment. The cover would be resistant to erosion, freezing and thawing cycles, wetting and drying cycles, and other sources of damage. It would be impermeable to precipitation.

Summary

A composite liner consisting of a 30-mil synthetic membrane placed between two 6-in. (152-mm)-compacted layers of soil-cement is a viable alternative for hazardous waste impoundments. It offers a high degree of reliability against seepage, is durable, and provides a firm working surface. The liner is constructed with conventional paving equipment. The in-place construction cost is competitive with comparable liners. It provides additional impoundment storage volume over conventional thicker lining systems in use today. Solid soil-cement can be effectively used for construction of hazardous waste impoundment dikes. This composite section can be considered as a final cover.

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LABORATORY EVALUATION OF SLURRY WALL MATERIALS OF CONSTRUCTION TO PREVENT CONTAMINATION OF GROUNDWATER FROM ORGANIC CONSTITUENTS

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Abstract

The installation of earthen liners and slurry wall trenches constructed of soils treated with bentonite and/or bentonite/cement mixtures are frequently used today as a means to retard or control the flow of contaminants from surface impoundments and landfill disposal areas. Although effective in reducing the rate of flow of water and some contaminants, standard bentonite treatment is not always effective in controlling some types of contaminants, such as chlorinated hydrocarbons.

Past handling and disposal of liquid chlorinated hydrocarbon waste in earthen impoundments and a landfill at an existing plant resulted in shallow ground water and soil contamination over a broad area. Following field investigations, extensive laboratory tests were conducted to investigate the feasibility of confining the plume of contamination through the installation of a slurry trench barrier. Specially designed methods of testing were used during the investigation.

Standard bentonite soil mixtures using up to 6% or more bentonite were shown to be ineffective in restricting the flow of the concentrated wastes found in the subsurface strata. However, a 12%/12% bentonite/cement dust soil mixture was shown to completely restrict the flow or movement of the concentrated waste while reducing the flow of ground water and contaminated leachate.

Introduction

Bentonite, or more specifically the clay mineral montmorillonite, has been used for decades to impede or reduce the flow of water through permeable soils. This characteristic has been widely used in the drilling and construction industries.

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The use of bentonite began in the petroleum exploration industry during the 1920's in drilling muds used to seal the sides of the hole and to provide other benefits which aided the drilling operation. During the 1940's, it was found that borings and trenches could be kept open using a slurry of bentonite, and that the resulting trench when backfilled exhibited a reduced permeability to the horizontal flow of ground water. More recently, bentonite slurry wall trenches have found application to control pollution migration for improperly designed waste landfills and impoundments.

The design of soil bentonite slurry walls for conventional groundwater control applications has been widely documented. Other than normal standard testing, very little investigative work is required. The design of a soil-bentonite slurry wall for waste containment however, requires site specific studies in order to determine the effectiveness of the proposed barrier.

Test Program

The presence of organic or inorganic compounds in the groundwater can have a detrimental effect on the bentonite slurry wall and its ability to contain contaminants. Certain chemicals can affect the chemical/physical properties of the bentonite and cause failure of the barrier either during construction or at sometime in the future. Therefore, the effects of the groundwater, waste leachate or liquid waste on the soil/bentonite system must be determined prior to the preparation of final design specifications for the slurry wall.

Prior to the start of laboratory investigations, detailed field investigations are essential to determine the geohydrology of the proposed slurry trench site. Unless detailed information is available, sufficient soil borings should be made of the entire area to define the geology, particularly within the area in which the proposed trench will be located. In addition, sufficient pump tests and field permeability tests should be made to define the hydrology of the area.

During the course of the field investigations, chemical analysis should be made on the groundwater and core samples to define the extent of the contamination plume. Samples of soil, the waste, ground water and leachate should be collected for use in subsequent laboratory evaluations of the proposed slurry trench composition.

In this paper, the authors describe the laboratory investigations conducted during the evaluation of a proposed slurry trench barrier to contain contamination from an existing industrial disposal area.

Site Description

The site for the proposed slurry trench barrier was an old disposal area within a large petrochemical manufacturing plant that had been used for various waste activities in the past. This included the collection and storage of liquid chlorinated hydrocarbon wastes in earthen pits or ponds and the landfill disposal of solid organic and inorganic wastes, drums of miscellaneous liquid organic wastes, plus construction debris such as wood, concrete, etc.

The field investigations indicated that extensive shallow groundwater and soil contamination had occurred. In addition, immicible and relatively pure undiluted waste organics were found to exist at points more than 300 feet away from the original source and down to depths as great as 50-60 feet below the surface.

Site Geology

The geology of the area was characterized by a combination of hydraulic and mechanical fill overlying deposits of recent alluvium. Extensive geological investigations of the area were conducted and a detailed description of this area developed.

The area where the proposed slurry trench barrier would be constructed was located on the eastern edge of the disposal area. Several core borings were drilled in the area where the proposed slurry trench was to be located.

The geology of the proposed slurry trench area was characterized by several distinct layers of soil. The area is covered with a layer of silty clayey sand fill material from the surface to a depth of approximately 12 to 14 feet below the surface. Beneath this on the northern end of the proposed trench area is a soft black organic clay layer approximately 4 to 6 feet thick. Beneath this is a layer of fine to medium silty sand approximately 25 to 35 feet thick that extended downward to a depth of 50 to 60 feet below the surface. Underlying these soil layers is a stiff light gray to tan clay found at depths of 50 to 60 feet below the surface throughout the area. Electric logs of wells in the area indicate the thickness of this base clay layer to be 75 to 100 feet.

Samples of soil and groundwater taken from core borings show the soil and water in the area of the proposed slurry trench to be contaminated with chlorinated hydrocarbons. Data from samples show contamination was present at various depths down to the base clay encountered at a depth of 57 feet below the surface. Although continuous samples were not taken for contamination analysis from all borings, a composite of samples taken at 2 feet intervals throughout the entire depth of the boring showed high levels of contamination was present at this location. Visual evidence of high concentrations of phased organic contaminants was encountered during the drilling operation, especially at depths of approximately 12 through 20 feet below the surface. An analysis of a composite of samples taken at 2 feet intervals throughout the depths of 2 feet to 65 feet below the surface showed the soil contained 0.59 wt.% total chlorinated hydrocarbons. These results are presented on Table 1. Groundwater samples taken from the same location continued 942 ppm of total chlorinated hydrocarbons as shown on Table 2.

Table 1 - Chemical Analysis of Composite Soil Mixture B

Compound	Weight Percent
Methyl Chloroform	0.02
Trichloroethylene	<0.01
Trichloroethane	0.03
Perchloroethane	0.04
Tetrachloroethane	0.01
Tetrachloroethane (Sym)	0.06
Pentachlorobutudiene	0.10
Hexachlorobutane	0.05
Hexachlorobutadiene	0.26
Unknown	0.02
Total Chlorinated Hydrocarbons	0.59

Table 2 - Groundwater Analysis

Component	Concentration ppm
Chloroethylene	0.80
Chloroethane	0.04
Methylene chloride	0.15
1,1 - Dichloroethene	25.9
1,1 - Dichloroethane	4.08
1,2 - Dichloroethene	--
Trichloromethane	--
1,2 - Dichloroethane	513
1,1,1, - Trichlorethane	4.82
Tetrachloromethane	--
Trichloroethene	7.12
1,1,2 - Trichlorothane	260
Tetrachloroethene	126
Total chlorinated hydrocarbons	942

Method of Test

A standard method of test for evaluating the inhibiting effects of soil additives to migration of organics through soils was not available. Therefore, modification of standard permeability testing of soils was required for this work.

A constant head permeability method of test was utilized throughout the laboratory investigation. However, instead of using the standard up-flow of fluid through the core to be tested, a downward flow was used in all cases. A flow diagram of the test apparatus is presented on Figure I.

The test apparatus was constructed from major component parts purchased from Soil Test, Inc. This included one high pressure K-670 Miniature Permeameter Unit plus two additional K-620 Soil Test Permeameters Units. These were connected in such a manner as to enable three separate permeability tests simultaneously. Subsequently, a second test unit consisting of two permeameters and pressure reservoir was added to expedite the test work with contaminated water as a fluid.

Operating pressures from 6 psi to 20 psi were used to accelerate the tests. Constant pressures were maintained through the use of an air compressor attached to the fluid reservoir. The volume of fluid flow was measured in the buret attached to each test unit (Figure 2), or by flowing into a graduated cylinder through the bottom drain valve.

Pretreatment of the soils with the additives to be tested was accomplished by direct addition on a weight percent basis. Approximately 100g of soil mixture was prepared for each test. Following the addition of the additives, water was added as required during mixing to achieve a medium to stiff consistency prior to compression in the test mold.

The unconsolidated soils were prepared for testing using parts from a Harvard Miniature Compaction Apparatus. The tare weight of the mold or sleeve from the permeameter plus a 200 mesh screen and a 3.0 cm porous stone was obtained. After clamping the mold in the mold holder, the screen was placed in the bottom of the mold. Approximately 30 cc of the soil to be tested was added and gently tapped or pressed into the mold. This amount of sample normally occupied approximately one-half of the total capacity of the mold. Water was added as required to maintain a saturated test core during compression. The porous stone was then placed on top of the test sample in the mold and pressure applied with the collar remover press. This unit is equipped with a spring loaded plunger with a base plate approximately 3.0 cm in diameter. The pressure was applied by pushing down on the compression rod. In total, approximately 50 pounds per square inch was applied for a period of 60 seconds. After compressing the sample into the mold, the mold was removed and the final weight of the unit with the sample recorded. Prior to assembling the mold in the permeameter the height and diameter of the test sample was measured and recorded.

Following assembly of the test units, water was added to the fluid reservoir. The resevoir was raised to the desired test pressure through the use of an air compressor. Prior to beginning the tests, sufficient water was added to each permeameter to displace the air trapped above the test core through the upper bleeder valve. Once the test unit was filled with water, the tests were begun by measuring the flow through the core at various intervals of time.

The organic permeability tests were also prepared as described above. Following measurement of the water permeability, the inlet line at the top of each unit was disconnected and excess water above the test cores removed (approximately 25 ml). Approximately 25 ml of chlorinated hydrocarbon waste was then added through the inlet opening at the top of each test unit. The water line was then reconnected and the test resumed by measuring the flow of water or organics at various time intervals. Following passage of the organic wastes through the test core, the tests were continued to measure the permeability of

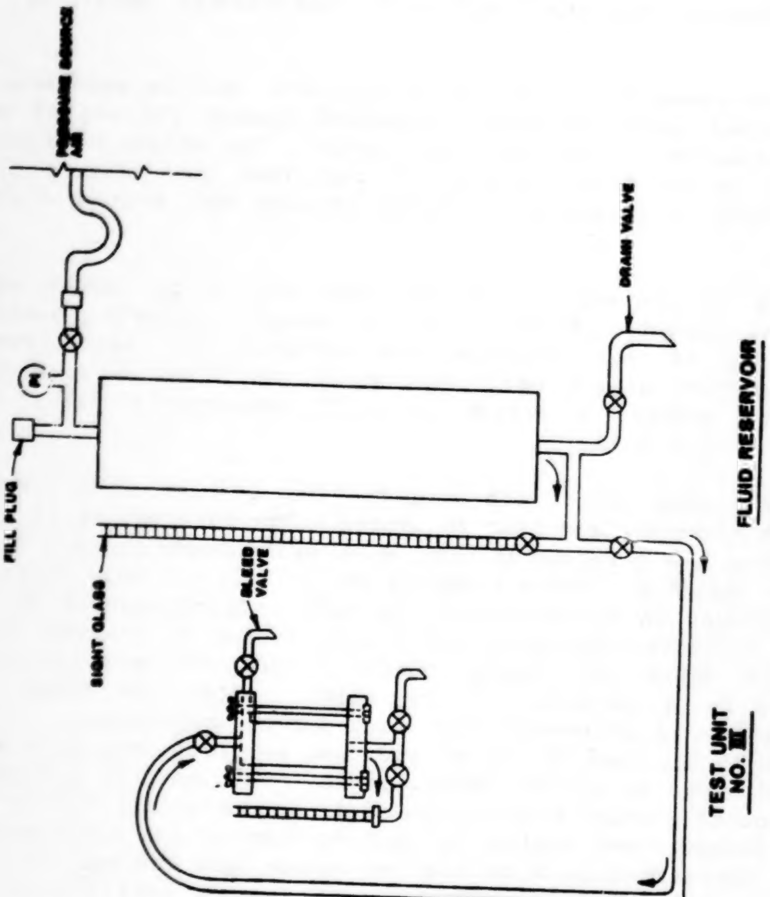

PRESSURE SOURCE AIR
FILL PLUG
PI
DRAIN VALVE
FLUID RESERVOIR
SIGHT GLASS
BLEED VALVE
TEST UNIT NO. III
TEST UNIT NO. II
DRAIN VALVE
TEST UNIT NO. I
FLOW BURET

FIGURE 1
PERMEABILITY TEST UNIT

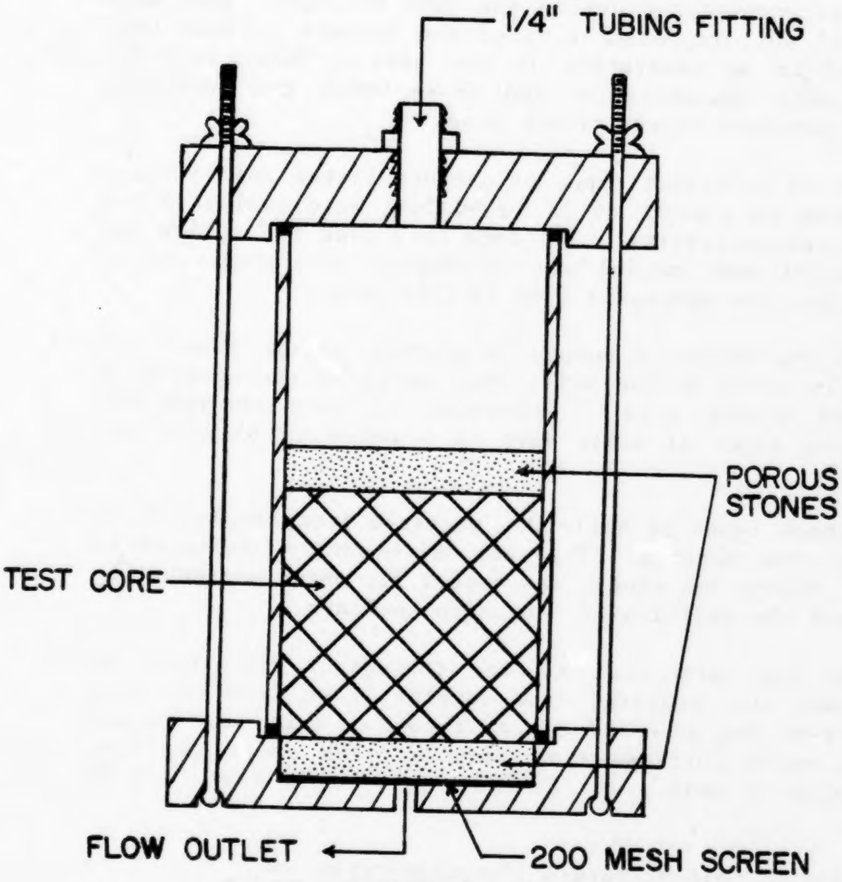

1/4" TUBING FITTING
POROUS STONES
TEST CORE
FLOW OUTLET
200 MESH SCREEN

FIGURE 2

TEST PERMEAMETER

water through the test core after exposure or contact with the organic waste.

Materials Tested

Soils Tested

Various soils were tested for water and/or organic permeability during the course of the study. Some of these soils were samples taken from specific stratas encountered during the core borings. Some were composite mixtures of soil representative of the overall stratas that would be encountered in an excavation in the area. These included soils free from organic contamination and those which are currently contaminated in the proposed slurry trench area.

Several samples of different types of uncontaminated soil stratas found in the area down to a depth of 55 to 60 feet were evaluated for water permeability characteristics. The data were used to confirm the accuracy of the special test method used throughout the study and to develop background data for hydraulic flow in this area.

In addition to the above, a composite mixture of uncontaminated soils was prepared in ratio to the soils that would be encountered in the proposed slurry trench area. According to core borings the percentage of various types of soils down to a depth of 64 feet are presented on Table 3.

A mixture of these types of soils was prepared from samples taken previously from the core borings. This composite sample, referred to as "Soil Mixture A" during the study, was tested for water permeability both with and without the addition of bentonite sealants.

A contaminated soil mixture, referred throughout the study as "Soil Mixture B", was also prepared from samples taken from the core borings in the area of the proposed slurry trench. This mixture was prepared by taking equal portions from samples taken every two feet during the drilling operations. The composition of soil mixture B is shown on Table 4.

Table 3 - Composition of Soil Mixture A (Uncontaminated Soil)

SOIL DESCRIPTION	DEPTH	PERCENTAGE
Fine to medium tan silty clayey sand	0 - 16	25.0%
Firm light gray and brown silty clay	16 - 19	3.7%
Soft to very soft black organic clay	19 - 23	6.2%
Light gray fine to medium sand	23 - 59	56.5%
Stiff light gray and tan clay	59 - 63	5.5%

Table 4 - Composition of Soil Mixture B (Contaminated Soil)

SOIL DESCRIPTION	DEPTH	PERCENTAGE
Tan silty clayey sand	0 - 12 ft.	20.8%
Tan and dark gray sandy silty clay	12 - 16 ft.	6.9%
Dark gray to black silty organic clay	16 - 20 ft.	6.9%
Silty, sandy clay	20 - 24 ft.	6.9%
Tan fine to medium sand	24 - 60 ft.	55.2%
Stiff gray and tan clay	60 - 62 ft.	3.4%

Fluids Tested

Four different fluids were utilized during the investigation. These included uncontaminated ground water, contaminated ground water, ground water which was saturated with chlorinated hydrocarbon wastes in the laboratory, and concentrated chlorinated hydrocarbon wastes.

Two five gallon containers of contaminated water from the area obtained for testing. An analysis of this water was previously presented in Table 2. This water sample was used throughout most of the test work and is referred to as water in the balance of the report. The water is of a quality that could be expected to flow through the proposed slurry trench.

The concentrated chlorinated hydrocarbon wastes used throughout the investigation was a sample of API separator organic waste from the plant. Although the exact composition of this material was not determined, it is believed that this is similar in composition to the waste materials processed in earthen ponds in the past that was the source of the plume of contamination in the area.

Additive Materials Tested

Several soil additive materials were evaluated during the course of the investigation. These included two bentonite materials and cement flue dust. Although other possible additive materials are available, the scope of the project was limited to the first material found to provide a successful barrier to the flow of the organic waste.

The bentonite materials used in the investigation were Volclay Bentonite 125 (BENT125) and Volclay Saline Seal 100 (SS100). These materials are manufactured by the American Colloid Company, Skokie, Illinois 60077. Bentonite 125 is specially formulated for use in slurry trench construction to reduce the flow of water. Saline Seal 100 is specifically formulated for use where the fluid to be controlled contains relatively high concentrations of salts and/or other contaminates.

The cement flue dust utilized through-out the investigation is a by-product manufactured by several cement plants located in Houston and Dallas, Texas. The sample of the cement flue dust tested was manufactured by Gulf Coast Cement in Houston.

Test Results

On-Site Soil Permeabilities

The water permeability of various uncontaminated soil strata found in the area was determined in the laboratory using the modified test method. These data are presented on Table 5. When compared with the field permeability data of similar type soils in the area, these data indicate the modified test method, used throughout the investigation produce results that are comparable with standard methods of test for field permeability measurements.

Several samples of contaminated soil from the borings were also tested for water permeability. These included stratas of soils encountered at various depths plus a mixtures of soils found at various depths in the area of the proposed slurry trench. The data from these tests are presented on Table 6.

The fill material (0-12 ft) above the contaminated area had a permeability of 5.0×10^{-5} cm/sec. A mixture of soil in the zone of highest contamination (12 to 25 ft) had a permeability of 2.1×10^{-7} cm/sec. Contaminated fine to medium sand (25 to 60 ft) had a permeability of 4.4×10^{-3} cm/sec, or approximately the same as the uncontaminated sand from the same strata. A mixture of contaminated and uncontaminated soils throughout a depth of 0 to 65 feet below the surface (Soil Mixture B) had permeabilities of 1.9×10^{-6} cm/sec and 2.4×10^{-6} cm/sec. The difference between the permeabilities of the contaminated soil mixture (Soil Mixture B) and the uncontaminated soil mixture (Soil Mixture A) of approximately 10^{-1} is probably a result of the contaminated soil mixture containing a higher percentage of sand (60% vs 56%) plus a lower percentage of dense clay (3.6% vs 5.5%).

Water and organic permeabilities were also determined on several samples of contaminated soils as shown on Table 7. The maximum permeability of Soil Mixture B to organics was 2.4×10^{-5} cm/sec as compared with an average initial water permeability of 2.7×10^{-6} cm/sec. The contaminated zone mixture had a maximum organic permeability of 3.1×10^{-5} cm/sec as compared with an average initial water permeability of 3.3×10^{-7} cm/sec. It is interesting to note that although the initial water permeabilities of these two soil mixtures differ by a factor of approximately 10, the organic permeability was approximately the same. An attempt was also made to determine the organic permeability of the stiff clay found at a depth of 60 feet below the surface. Although the clay exhibited an initial permeability of 1.8×10^{-8} cm/sec to water and continued to flow water throughout the test, there was no visible flow of organic through the test core as experienced during the other soil tests.

Table 5 - Water Permeability of Uncontaminated Soils

Sample Description (Source)	Permeability, cm/sec,
Fill: Tan & gray silty with sand pockets and organic matter (5-7 feet)	2.3×10^{-4}
Fill: Tan & light gray clayey sand (3-5 feet)	5.0×10^{-5}
Soft dark gray organic clay (20-22 feet)	9.0×10^{-5}
Fine gray sand (50-52 feet)	2.0×10^{-3}
Stiff light gray and tan silty clay (62-63 feet)	1.2×10^{-8}
Soil Mixture (Composite of uncontaminated soils found in the area from 0-63 feet below surface)	3.7×10^{-7} 1.9×10^{-7}

Table 6 - Water Permeability of Contaminated Soils

Core Description	Permeability cm/sec,
Fill: Silty, clayey sand (0 to 12 feet)	5.0×10^{-5}
Silty, sandy clay* (12 to 25 feet)	2.1×10^{-7}
Medium tan to light gray sand (25 to 60 feet)	4.4×10^{-3}
Stiff light gray clay (62 to 65 feet)	1.8×10^{-8}
(Composite of Soil Mix B contaminated soils from 0-62 feet below the surface)	2.4×10^{-6} 1.9×10^{-6}

*Zone of highest concentration of contamination

Table 7 - Permeability of Contaminated Soils

Soil Sample	Permeability, cm/sec		
	Initial Water	Organic	Final Water
Soil Mixture B (0-62 feet)	2.7×10^{-6}	2.4×10^{-5}	9.0×10^{-7}
Contaminated Zone (12-26 feet)	3.3×10^{-7}	3.1×10^{-5}	2.0×10^{-7}
Base Clay (60-62 feet)	1.8×10^{-8}	(No organic flow)	4.1×10^{-8}

Evaluation of Bentonite 125 and Saline Seal 100

A series of water permeability tests were conducted on samples of contaminated Soil Mixture B when treated with from 1% to 6% Bentonite 125 or Saline Seal 100. These tests were conducted to compare the performance of the two different bentonite materials in the presence of the contaminants contained in the on-site soil. The results from these tests are presented graphically on Figure 3.

The data from these tests show the addition of increasing amounts of bentonite to Soil Mixture B results in a gradual decrease in the permeability of the soil to water. With 6% bentonite, the permeability was reduced from approximately 2×10^{-6} cm/sec to 3×10^{-7} cm/sec. Very little if any difference was observed in the performance of the Bentonite 125 and the Saline Seal 100 with the contaminated soil. However, Saline Seal 100 was selected for use in all subsequent test work due to its reported resistance to high concentrations of salts.

Permeability of Bentonite Treated Contaminated Soil

A series of tests were run to determine the water and organic permeability of the contaminated in-place soils (Soil Mixture B) when treated with up to 16% Bentonite 125 or Saline Seal 100.

When treated with increasing amounts of Bentonite 125, a proportional decrease in the permeability of Soil Mixture B to water and organics was observed. The average permeability to water was reduced from approximately 2.7×10^{-6} cm/sec down to 7.9×10^{-9} cm/sec with the addition of 16% Bentonite 125. The maximum permeability to organic was reduced from 2.4×10^{-5} cm/sec down to approximately 1.3×10^{-6} cm/sec at a 16% Bentonite 125 treatment level. In most instances, the permeability to water was shown to be approximately the same as it was originally following passage of the organics through the test core. A plot of the permeabilities vs percent Bentonite 125 is presented in Figure 4.

Treatment with increasing amounts of Saline Seal 100 resulted in similar reductions in the permeabilities. To water, the average permeability was reduced from 2.7×10^{-6} cm/sec down to approximately 9.0×10^{-9} cm/sec with 16% SS100. The maximum permeability to organics was reduced from 2.4×10^{-5} cm/sec down to approximately 1.1×10^{-7} cm/sec at the 16% treatment level. A plot of the permeabilities vs percent SS100 is presented in Figure 5.

The data from these tests indicate treatment of a mixture of in-place soils in the area of the proposed slurry trench would require the addition of a minimum of 12% bentonite (Bentonite 125 or Saline Seal 100) to achieve a permeability of 1×10^{-7} cm/sec to water. Treatment with a minimum of 16% bentonite (Saline Seal 100) would be required to achieve a permeability of approximately 10^{-7} cm/sec or less to organics. Although limited data are available on a 16% Bentonite 125 treated soil, Saline Seal 100 appears to be slightly more efficient in reducing the permeability of the organic. At all

FIGURE 3

WATER PERMEABILITY vs PERCENT BENTONITE

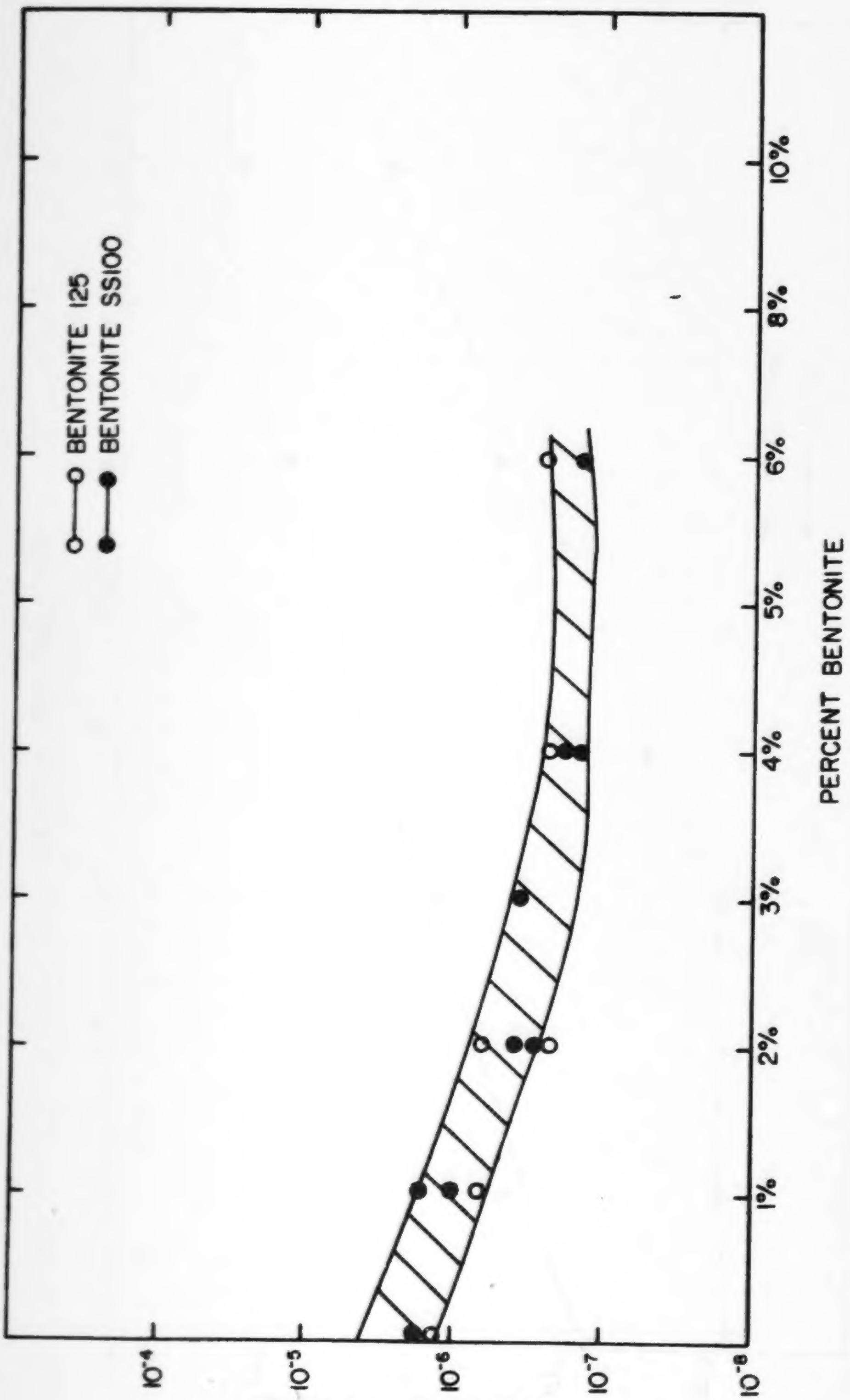


FIGURE 4

PERMEABILITY vs PERCENT BENTONITE 125

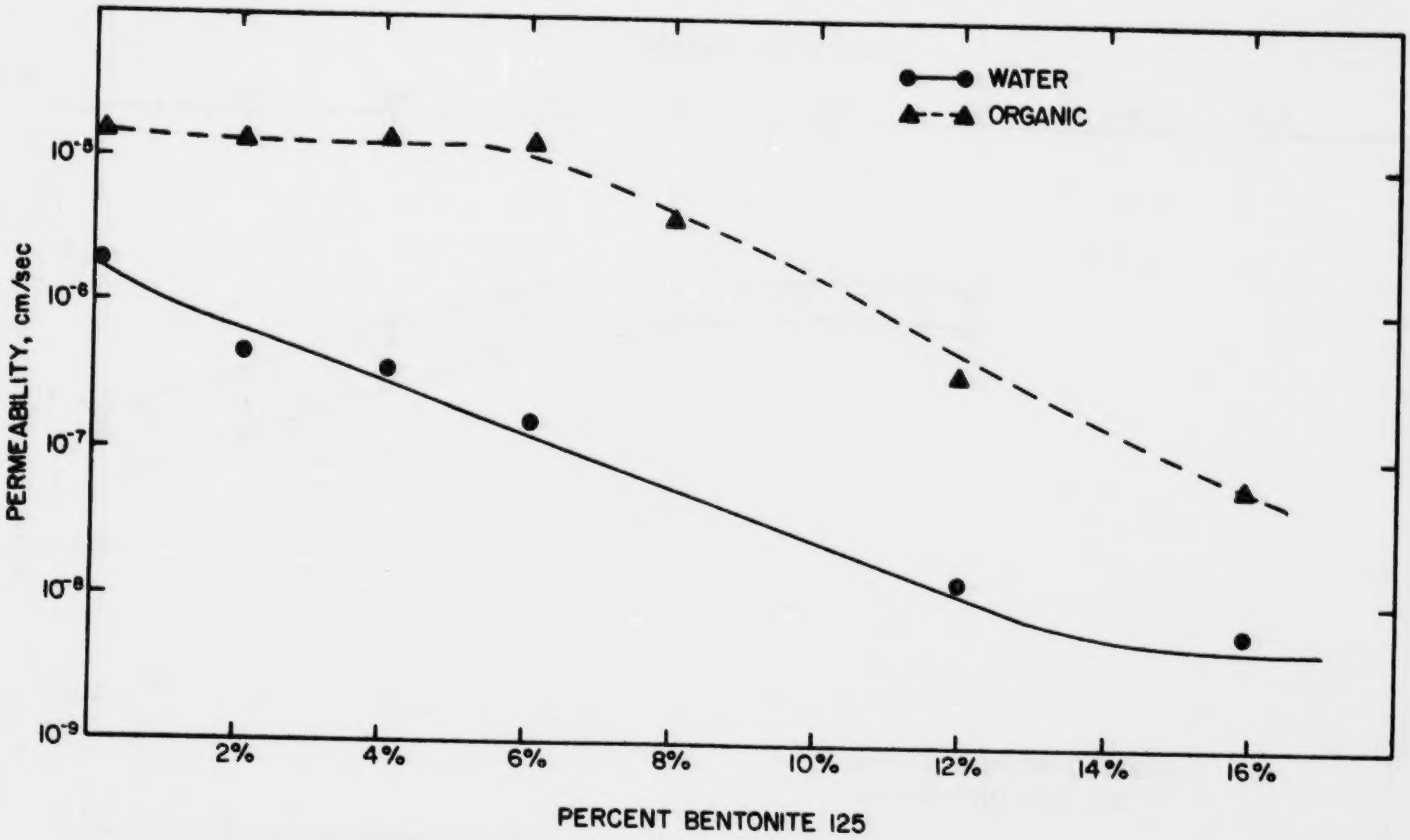
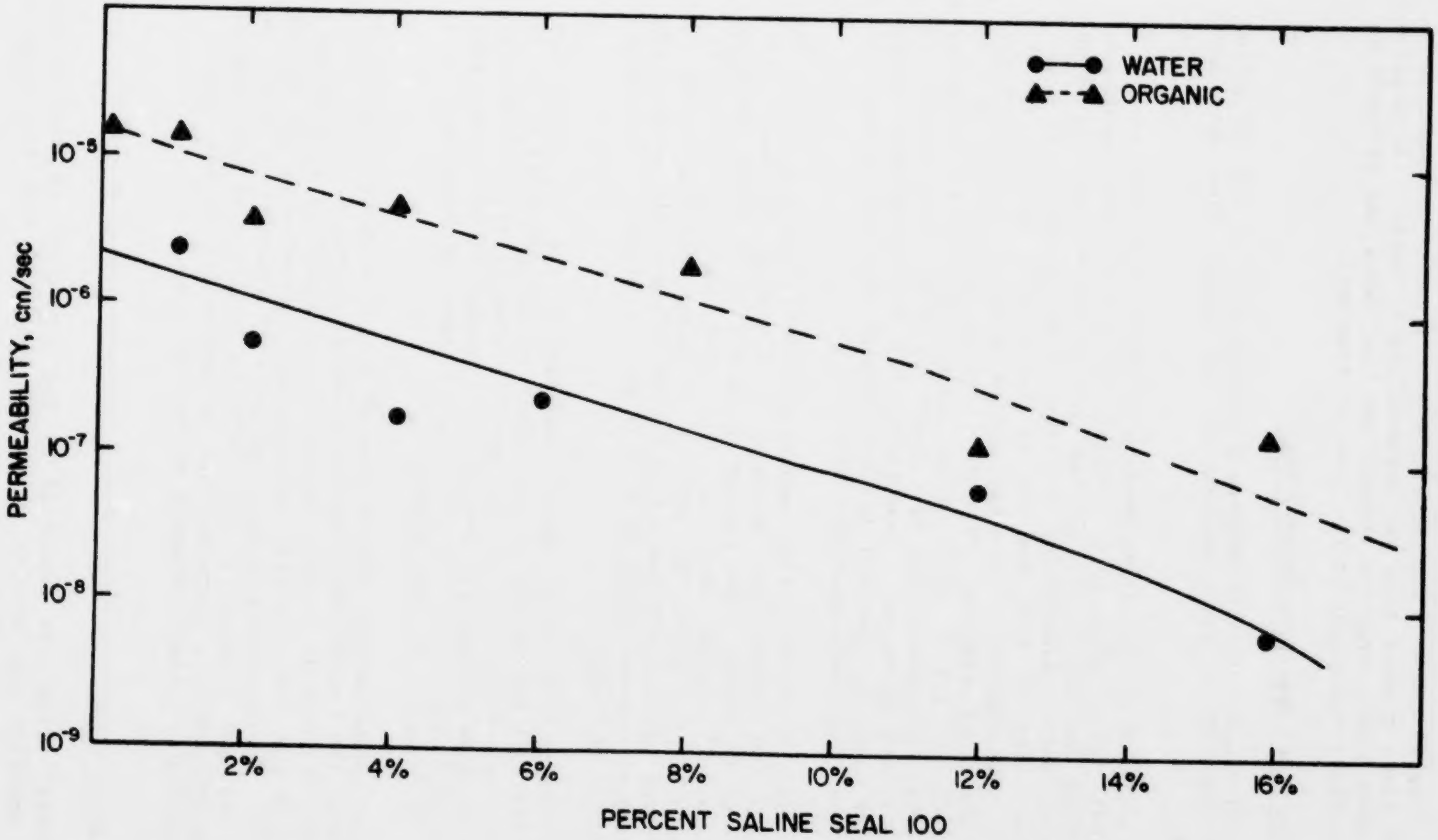


FIGURE 5

PERMEABILITY vs SALINE SEAL 100



treatment levels, however, the permeability to organics was shown to be 10 to 100 times greater than the permeability to water. A positive flow of concentrated organics through the test cores was observed in all of the tests regardless of the level of treatment.

Cement Flue Dust/ Bentonite Treated Soil

Several samples of Soil Mixture B treated with 6% and 12% Saline Seal 100 plus 1% up to a maximum of 18% cement flue dust were evaluated.

The addition of increasing amounts of cement flue dust to the 6% Saline Seal 100 treated soil resulted in a relatively small but corresponding increase in the permeability of the mixtures to water. The average water permeability increased from approximately 3.8 x 10^{-7} cm/sec with no flue dust up to 5.6 x 10^{-7} cm/sec with 12% flue dust. The addition of flue dust failed to reduce the permeability of the mixtures to organics. Permeabilities within the range of 1.7 x 10^{-6} cm/sec to 1.9 x 10^{-5} cm/sec were measured for organics. In all cases, the permeability to organic was shown to be higher than the permeability to water. Essentially all of the organic charged during each test was observed to pass through the test cores. The results from these tests presented on Figure 6.

The addition of increasing amounts of cement flue dust to the 12% Saline Seal 100 treated soil resulted in a similar increase in the peremabiilty to water. The average permeability to water increased from 1.7 x 10^{-8} cm/sec with no flue dust up to 9.5 x 10^{-7} cm/sec with 18% flue dust. The addition of cement flue dust to the 12% SS100 treated soil however had a significant effect on the permeability to organics at concentrations of 6% flue dust or greater. The results are presented on Figure 7.

The addition 1% to 6% cement flue dust to the 12% Saline Seal 100 treated soil had little if any effect on the permeability to the organics. Essentially all of the organics charged to the units was observed to flow through the test cores. Permeabilities in the range of 1 x 10^{-7} cm/sec to 5 x 10^{-7} cm/sec were measured for water and 5 x 10^{-6} cm/sec for the organic.

Using concentrations greater than 6% flue dust mixed with the 12% Saline Seal 100 treated soil, the addition of organics to the system ultimately resulted in a sharp decrease in permeability with ultimate plugging of the core. Similar results were obtained with 12% and 18% flue dust. In most cases a small quantity of water (1 to 3 ml) was observed to pass through the core following addition of the organic. This was followed by a sharp decrease in permeability to less than 1 x 10^{-9} cm/sec with ultimate plugging of the test core.

Confirmation Tests

Several tests were conducted in the laboratory to confirm the initial positive test results on the 12% SS100/12% flue dust treated

FIGURE 6
PERMEABILITY vs FLUE DUST
(WITH 6% SLSIOO TREATED SOIL)

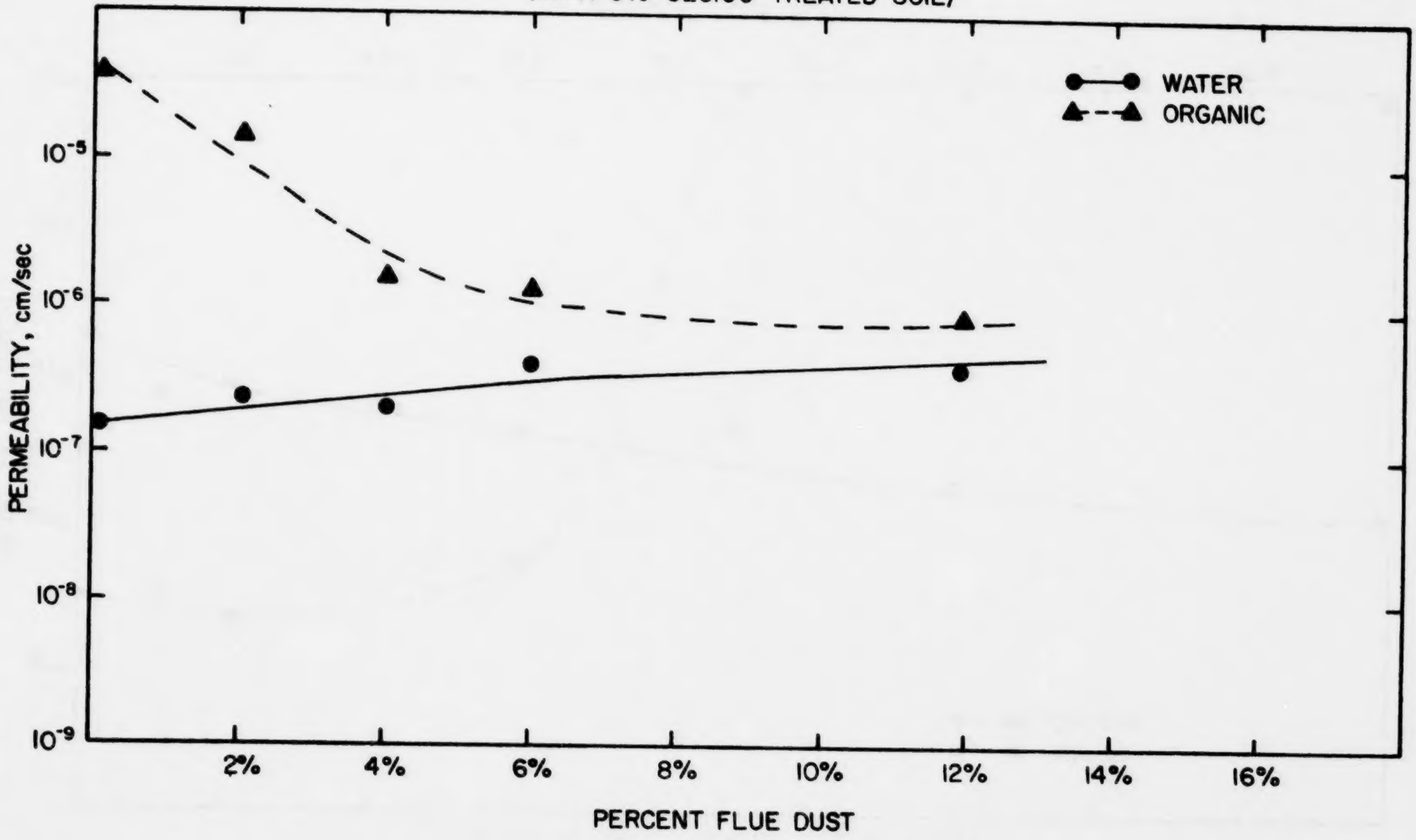
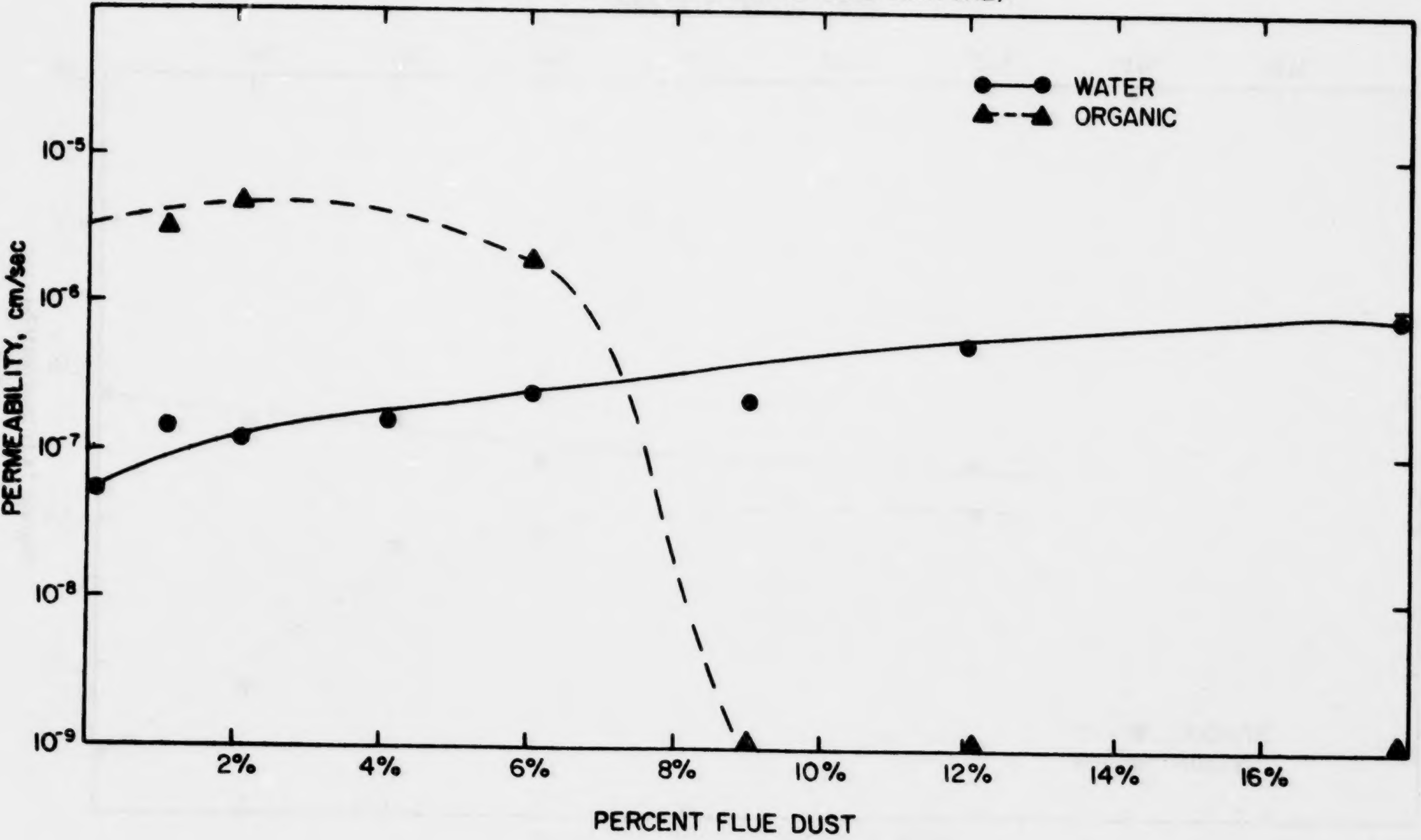


FIGURE 7
PERMEABILITY vs PERCENT FLUE DUST
(WITH 12% SS100 TREATED SOIL MIXTURE)



soils. Tests were also run to investigate the effects of various other factors on the efficiency and long term stability of these systems. In addition, a test was fun to confirm the initial results obtained on the stiff clay that unerlies the area.

Permeability of 12% SS100/12% Flue Dust Treated Soil

A sample of Soil Mixture B treated with 12% SS100 and 12% flue dust was prepared and tested in accordance with the standard procedures used throughout the investigation. The resulting mixture exhibited an average permeability of 6.1 x 10^{-7} cm/sec to water. Upon contact with the organic waste, a sharp decrease in permeability was observed that ultimately resulted in plugging (permeability less than 1 x 10^{-9}) cm/sec of the core following passage of approximately 2.5 ml of water.

12%/12% Mixture with Contaminated Water

A sample of Soil Mixture B treated with 12% flue dust and 12% Saline Seal 100 was prepared for testing using contaminated water for hydration of the mixture. This test core exhibited an average permeability to water of 8.1 x 10^{-7} cm/sec. Following addition of organics to the unit, the permeability increased to approximately 1.1 x 10^{-6} cm/sec for the next 5 ml of fluid to pass through the core. During this period, approximately 1.5 ml of the 25 ml of organic charged to the unit passed through the core. Subsequently, a sharp decrease in permeability occurred with ultimate plugging and no flow of fluid through the core.

12% Flue Dust/12% SS100 Time-Set Compaction

A sample of Soil Mixture B treated with 12% flue dust and 12% Saline Seal 100 was prepared for testing. In this test, the mold was filled with contaminated water and allowed to cure over a period of 12 days prior to beginning the permeability testing. This core exhibited a permeability of 5.2 x 10^{-7} to water. Contact with the organics resulted in a sharp decrease in permeability with plugging of the core following passage of approximately 3 ml water. No flow of organic through the core was observed during the test.

Water Stripping of Available Alkalinity

Analysis of the effluent water from previous tests using cement flue dust indicated an increase in pH from 7.6 up to 12.0 could be expected due to leaching of the available alkalinity in the flue dust from the treated soil mixture. A test was therefore begun to evaluate the long-term effects of water stripping of the available alkalinity from the 12%/12% system on the efficiency of the system to retard or prevent the movement of phased organics.

A sample of Soil Mixture B treated with 12% flue dust and 12% SS100 was prepared in accordance with the standard procedure for testing. The pH of the feed water and the effluent was 6.9 and 12.0, respectively at the start of the test. A gradual decrease in the pH of the effluent occurred during the course of the test. After three months, 7,300 ml of water had passed through the core and the pH of the effluent had decreased to 7.0. At that time, 22 ml of organics were charged to the unit.

The permeability of the test core to water remained relatively constant at approximately 3 x 10^{-7} cm/sec during the first 2,000 ml of water to pass through the core. The pH of the effluent at this point had decreased to approximately 9.8. A gradual increase in permeability to approximately 3 x 10^{-6} cm/sec occurred during passage of the next 5,300 ml of water. Following the addition of the organics, permeabilities within the range of 1.0 x 10^{-6} cm/sec to 6.0 x 10^{-6} cm/sec were measured for the next 30 ml of fluid to pass through the core. This flow consisted of approximately 20 ml of phased organics and 10 ml of water. Subsequentely, the flow consisted of water free from phased orgaincs at a permeability of 5 x 10^{-6} cm/sec for the balance of the test. The pH of the effluent water at the end of the test was 6.9.

Effects of Contaminated Water on the 12%/12% Mixture

A sample of Soil Mixture B treated with 12% flue dust and 12% Saline Seal 100 was prepared in accordance with the standard procedure for testing. Saturated contaminated water prepared in the laboratory were used to determine the permeability to water. Approximately 2,000 ml of contaminated water was passed through the test core prior to contact with the concentrated organic waste. During this period, the pH decreased from 11.9 down to approximately 8.3.

An average permeability of 3.0 x 10^{-7} cm/sec was measured for contaminated water. On contact with concentrated organic, a sharp decrease in permeability was observed following passage of an additional 2.5 ml of water down to approximately 1 x 10^{-9} cm/sec. No phased organic passed through the core during the test.

Base Clay Permeability

An undisturbed sample of the stiff gray base clay that is found throughout the area was prepared for long-term testing for water and organic permeability. The test core exhibited an average permeability to water of 4.9 x 10^{-9} cm/sec. Following passage of 4.1 ml of water through the core, 25 ml of organic was added to the unit. During the next 80 days approximately 25 ml of additional water passed through the test core with no visible organics in the effluent. An initial increase in permeability to approximately 1 x 10^{-8} cm/sec was measured immediately after the organic was added, followed by a decrease to within the range of 5 x 10^{-10} cm/sec and 6 x 10^{-9} cm/sec during the balance of the test.

Discussion of Results

The permeability of concentrated or phased chlorinated hydrocarbon wastes through the type of soils found above the stiff base clay in the area was shown to be substantially greater than the permeability of water through the same soils. A mixture of soils in the zone of maximum contamination (12 to 25 feet below the surface) in the area of the proposed slurry trench exhibited a permeability of 3.3 x 10^{-7} cm/sec to water and 3.1 x 10^{-5} cm/sec to the concentrated organic waste. A mixture of soil found from the surface down to a depth of 62 feet had a permeability of 2.6 x 10^{-7} cm/sec to water and 2.4 x 10^{-5} cm/sec to organic.

The stiff gray and tan base clay that extends throughout the area which was encountered at a depth of approximately 50 to 60 feet below the surface in the area of the proposed slurry trench was shown to be an effective barrier to the downward migration of the waste. This stiff clay was shown to have a permeability in the range of 4.9 x 10^{-9} cm/sec and 1.8 x 10^{-8} cm/sec to water. The clay was impermeable to the flow of the concentrated or phased chlorinated hydrocarbon.

Treatment of the in-place soils to produce an effective slurry trench barrier to the migration of the phased organic waste was shown to be feasible using one or two different compositions. In-place contaminated soils when treated with 16% Saline Seal 100 exhibited a permeability of approximately 9 x 10^{-9} cm/sec to water and 3 x 10^{-7} cm/sec to the concentrated organic. In-place soils treated with 12% cement flue dust and 12% Saline Seal 100 exhibited a permeability of approximately 4 x 10^{-7} cm/sec to water but was shown to be essentially impermeable to the concentrated organic waste. On contact with the organic, a sharp decrease in permeabiilty occurred and ultimately after passage of approximately 1-2 ml of water, the test cores plugged with essentially no further flow of water or organics.

The 12% flue dust/12% Saline Seal 100 treated soil mixture was shown to have a limited effective life time to prohibit or restrict the flow of phased organics. Water flowing through the mixture (permeability in the range of 3 x 10^{-7} cm/sec to 4 x 10^{-6}) cm/sec resulted in stripping of alkaline constituents from the system and subsequent failure to restrict the flow of phased organics. According to the test data available from the contaminated water flow test and the long term water stripping test failure should occur following passage of from 73 to 225 trench volumes of water through the barrier. Under normal conditions, the amount of time required for this volume of water to move through the slurry wall would range from approximately 1000 to 3000 years.

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Ground Water Quality Management At A Concentrated Waste Site

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INTRODUCTION

In May 1983, Pinellas County's 2100 ton per day Refuse to Energy Facility (REF) commenced full-scale operation. The facility is located on a 700 acre site in the most densely populated county in Florida (Figure 1).

The REF site has been the historical "dumping ground" for the region for the past 75 years. Many active and closed landfills dot the landscape. In addition, a portion of the site has been utilized for the land application of domestic wastewater sludges.

Since the plant began start-up procedures, additional waste flows have been concentrated at the site. Specifically:

1. High quality tertiary effluent from the Pinellas County system is employed as cooling water at the REF.
2. In the very near future, all municipal sludges and commercial grease will be transported to a fluidized bed incinerator to be located on-site. The incinerator will be fired by landfill gas extracted from closed landfills on and adjacent to the REF site.
3. The incinerator produces a residual material, the bulk of which is called aggregate. This aggregate is used at site landfills.
4. Due to recent state legislation, a hazardous waste transfer station is under consideration.

The REF site is therefor considered a concentrating point for a wide variety of wastes and component residuals. Due to the sever hydrogeologic setting and stringent ground water regulations, innovative control measures are required.

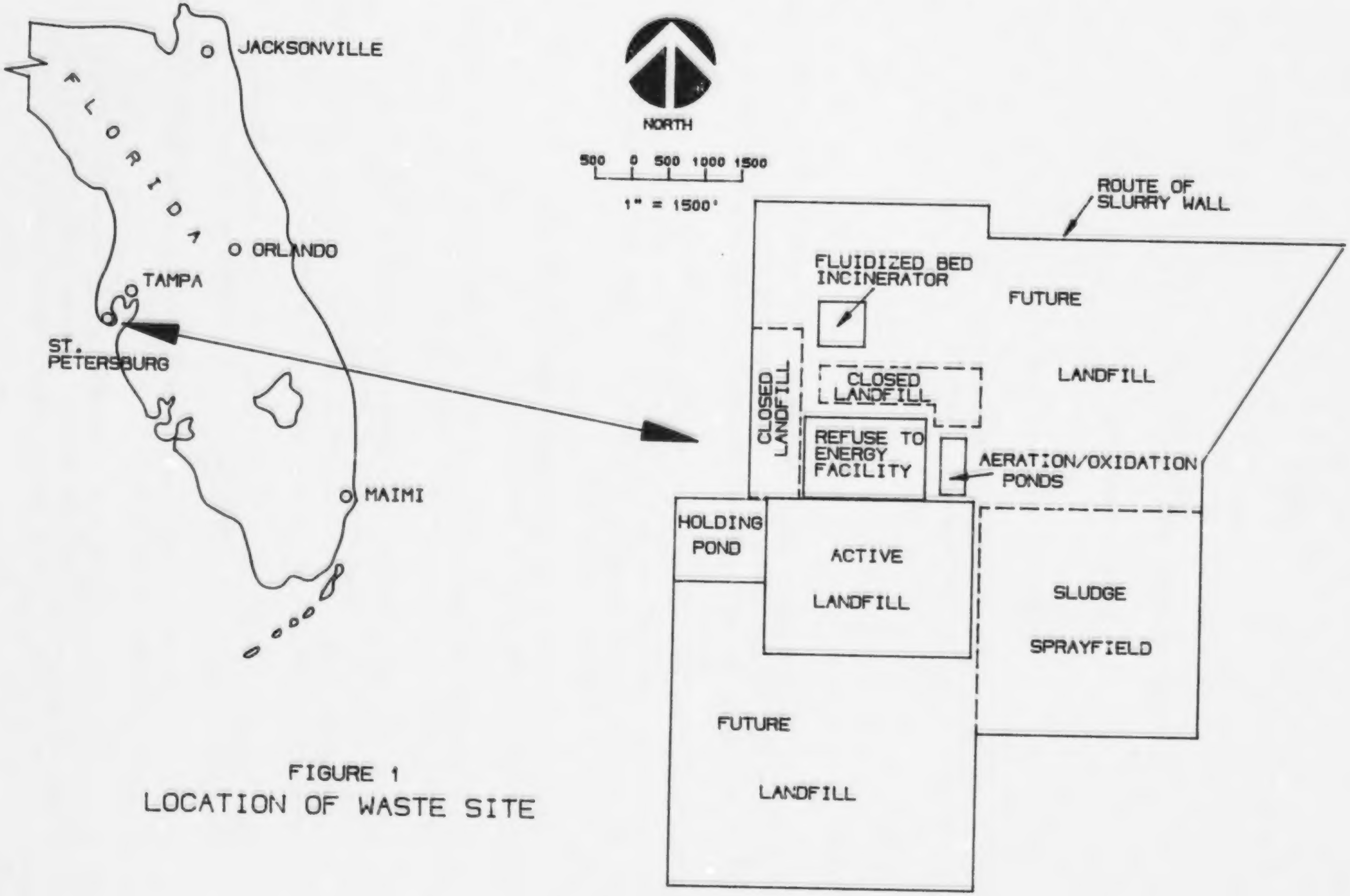

JACKSONVILLE
FLORIDA
ORLANDO
TAMPA
ST. PETERSBURG
MAIMI
NORTH
500 0 500 1000 1500
1" = 1500'
ROUTE OF SLURRY WALL
FLUIDIZED BED INCINERATOR
FUTURE LANDFILL
CLOSED LANDFILL
CLOSED LANDFILL
REFUSE TO ENERGY FACILITY
AERATION/OXIDATION PONDS
HOLDING POND
ACTIVE LANDFILL
SLUDGE SPRAYFIELD
FUTURE LANDFILL

FIGURE 1
LOCATION OF WASTE SITE

ABSTRACT

The 2100 ton/day Pinellas Resource Recovery Facility lies adjacent to active and closed landfills, and a land application site for sludges. Surface water and ground water in the shallow aquifer are virtually indistinguishable; the water table is near land surface much of the year. An aquiclude, which separates the shallow aquifer from the limestone Floridan aquifer underlies the entire area at an average depth of 90 feet. The aquiclude ranges in thickness from 15 to 80 feet and in conductivity from 10^{-6} to 10^{-9} cm/sec.

The State of Florida has the most stringent ground water regulations in the nation. Ground water at the site boundary must meet the Federal Safe Drinking Water Standards (SDWS). In addition, stormwater discharges are subject to strict quality criteria. A comprehensive surface/ground water monitoring network is now in place.

Background water quality in the shallow aquifer indicates that the SDWS are not being met. Iron, color and turbidity values are elevated due to natural causes. Water from wells down gradient from old landfills often contains high concentrations of chromium, lead, and aluminum. Ground water beneath the sludge application site is contaminated by nitrates. Stormwater quality is similarly deteriorated; high concentrations of lead, ammonia, and cadmium are common.

The strategy for compliance with these regulations is composed of conceptual and technical solutions. The first step is to organize the pollution sources under a single site permit. Step two is to construct a slurry wall around the entire site which is keyed into the aquiclude. Step three is to install a drainage system within the wall which conveys water to the plant's cooling system. The final step is to upgrade the site's aeration basin to accept collected leachate.

HYDROGEOLOGY

Site geology is very characteristic of coastal Florida landforms (Figure 2). It is Karst terrain, with a sandy clay overburden worked by recent sea level changes. The "basement" rocks underlying the site are limestones of Oligocene, Eocene, and Miocene ages. The limestones lie unconformably below an aquiclude at a typical depth of 80 to 94 feet below land surface. The Hawthorne Formation, a clayey limestone of Miocene age, forms an aquiclude between the underlying limestones and the surficial sand and shell deposits. The surficial overburden contains deposits of Pleistocene and Pliocene age. They are primarily composed of quartz sands, intermixed with clays and shell. Phosphatic pebbles and chert may also be present. In general, the clay content of soils increases with depth. Layers of shell, impregnated with silt; form discontinuous beds at shallow depths below land surface.

There are no surficial outcrops of limestone at the site. Relic sinkholes, now filled with densely compacted clayey sand, dot the terrain. The depth to limestone formations in these zones is usually greater than 100 feet below grade.

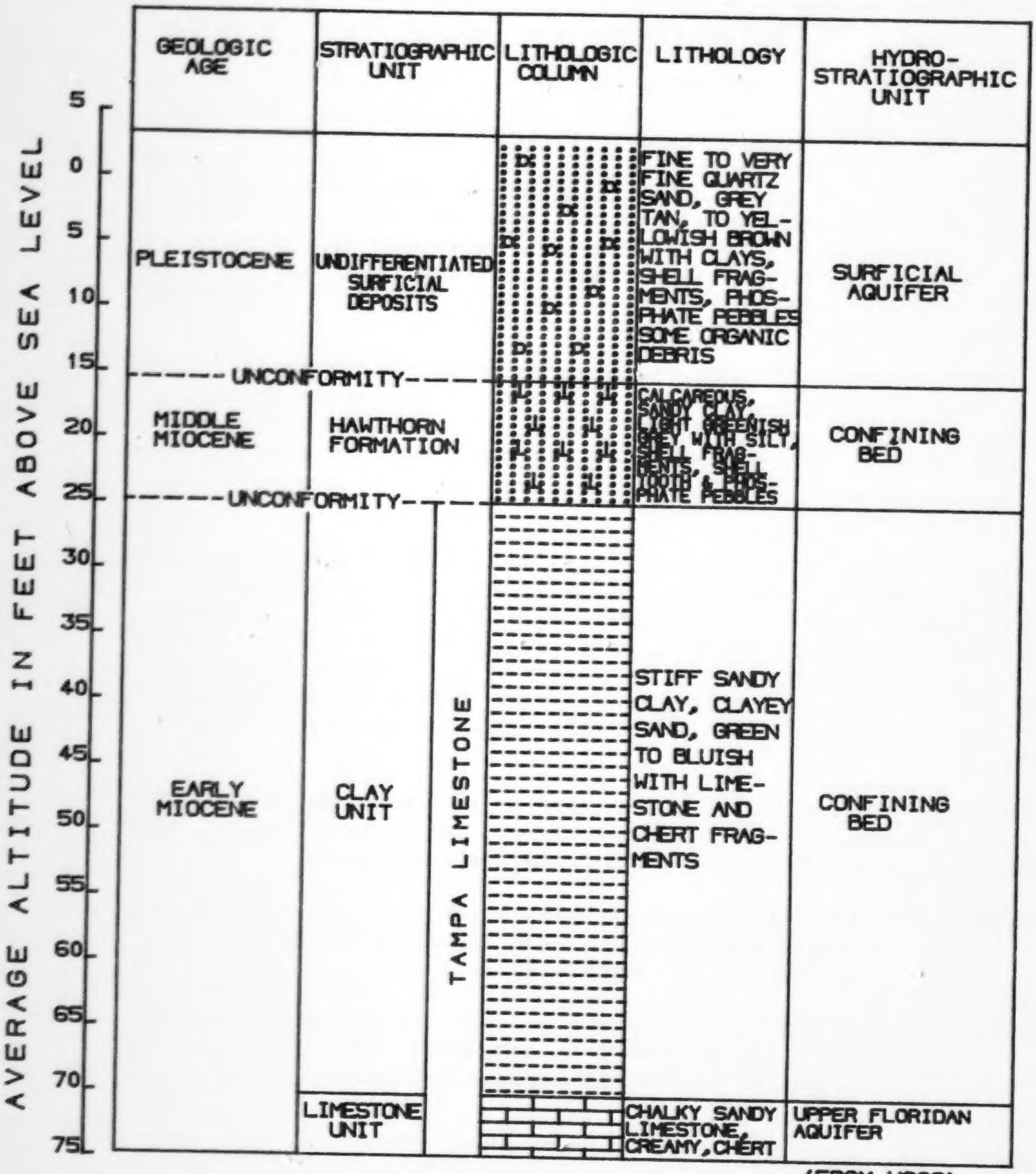
The Hawthorne Formation is undoubtedly the most significant stratum at the REF for the following reasons:

1. It provides a physical barrier to vertical exchange of ground water in the surficial sands and the limestone;
2. Its presence provides the seal necessary to impart artesian characteristics to the ground water in the limestone formations, commonly known as the Floridan aquifer;
3. Its presence controls the characteristically high water table at the REF in the surficial, shallow aquifer; and
4. It provides one important component of the proposed method of leachate control at the site.

The aquiclude is composed of clayey fine sand,sandy clay, and clay; it underlies the site at a typical depth of 18 to 22 feet below grade. It varies in thickness; a typical range being between 60 and 80 feet, with a minimum reported value of 15 feet. The layer "stiffens" with increasing depth; blow counts greater than 100 are common. In addition, the upper 10 feet of this layer may be impregnated with bits of calcareous materials.

Based on undisturbed samples, taken at the site, the Hawthorne Formation has a coefficient of permeability ranging from 10^{-6} to 10^{-9} cm/sec. The gradient through the aquiclude, between the water table aquifer and the artesian Floridan aquifer, is downward. On the average, there is approximately 1.0 foot of driving head, yielding a gradient of 0.01 Ft/Ft. Assuming uniform, steady state conditions, the downward migration of surficial ground water into the Floridan aquifer travels at a rate of 0.01 Ft/yr.

Horizontal flow is the governing parameter for ground water in the shallow


AVERAGE ALTITUDE IN FEET ABOVE SEA LEVEL
5
0
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65
70
75
GEOLOGIC AGE
STRATIOGRAPHIC UNIT
LITHOLOGIC COLUMN
LITHOLOGY
HYDRO-STRATIOGRAPHIC UNIT
PLEISTOCENE
UNDIFFERENTIATED SURFICIAL DEPOSITS
FINE TO VERY FINE QUARTZ SAND, GREY TAN, TO YELLOWISH BROWN WITH CLAYS, SHELL FRAGMENTS, PHOSPHATE PEBBLES SOME ORGANIC DEBRIS
SURFICIAL AQUIFER
UNCONFORMITY
MIDDLE MIOCENE
HAWTHORN FORMATION
CALCAREOUS, SANDY CLAY, LIGHT GREENISH GREY WITH SILT, SHELL FRAGMENTS, SHELL TOOTH & PHOSPHATE PEBBLES
CONFINING BED
UNCONFORMITY
EARLY MIOCENE
CLAY UNIT
TAMPA LIMESTONE
STIFF SANDY CLAY, CLAYEY SAND, GREEN TO BLUISH WITH LIMESTONE AND CHERT FRAGMENTS
CONFINING BED
LIMESTONE UNIT
CHALKY SANDY LIMESTONE, CREAMY, CHERT
UPPER FLORIDAN AQUIFER

(FROM USGS)

FIGURE 2

GEOLOGIC FRAMEWORK

aquifer. Slug tests performed in shallow aquifer wells indicate flow velocities in the 0.1 to 10 Ft/year range. The controlling factor of flow velocity is the magnitude of the gradient, since the porosity and conductivity of the surficial deposits are fairly uniform. For example, in terrain where no surface ditches or borrow pits are found, flow is sluggish. However, as a canal or ditch is approached, flow accelerates by several orders of magnitude. This points out the important relationship between surface water at the site and the ground water in the surficial aquifer. From an engineering standpoint, they are indistinguishable. Specifically, the water table is at or near the land surface during most of the year. Excavations, such as ditches, borrow pits, and canals alter the regional flow pattern of ground water in the surficial aquifer, with water now flowing to the excavation. This makes the site amenable to dewatering by lateral ditch methods. However, the flow of water into the ditches from the surficial aquifer is limited by the rate at which surface water is discharged from the site via evapotranspiration, stormwater structures, or leakage through the confining layer.

In the vicinity of the REF site there are no potable water wells in either the surficial or Floridan aquifers. Water yields from wells in the surficial aquifer are minimal; typical flow rates are less than 5 gpm. The ground water in the Floridan aquifer is too saline for potable use.

GROUND WATER REGULATIONS IN FLORIDA

Approximately 90% of Florida's population obtains their potable water from ground water supplies. For years the use of certain land practices, such as sewage lagoons, septic tanks, underground storage tanks, and agricultural spraying remained unchecked. Because of the high porosity of surficial soils, degradation of the quality of the potable supplies has occured. In the past few years, the State of Florida has implemented stringent ground water regulations aimed at protecting this critical resource.

Of foremost importance, the Ground Water Quality Act of 1983 established a new classification scheme for ground waters. Each classification is accompanied by a set of minimum criteria as described below:

CLASS G-I	Potable water use, ground water in single source aquifers which has a total dissolved solids content of less than 3,000 mg/I.
CLASS G-II	Potable water use, ground water in aquifers which has a total dissolved solids content of less than 10,000 mg/I, unless otherwise classified.
CLASS G-III	Non-potable water use, ground water in unconfined aquifers which has a total dissolved solids content of 10,000 mg/I or greater, or which has total dissolved solids of 3,000-10,000 mg/I and either has been reclassified as having no reasonable potential as a future source of drinking water.

CLASS G-IV Non-potable water use, ground water in confined aquifers which has a total dissolved solids content of 10,000 mg/I or greater.

The ground water at the REF site, and the majority of the rest of the State, is classified as G-II. Among the many standards which apply to these waters is the provision that the water quality of G-II ground water will not exceed the Safe Drinking Water Standards.

Recognizing that many G-II ground waters may not meet these criteria due to existing surface land usage (such as landfills), the State has established a Zone of Discharge (ZOD) concept. A ZOD is analagous to a mixing zone for effluent discharges to surface waters. For example, a landfill operator may be granted a ZOD beneath the landfill site which extends out peripherally to the site boundary, or 100 feet from the landfill cell, whichever is more restrictive. Within the ZOD, the Safe Drinking Water Standards do not have to be met. However, ground water in a ZOD must be free from pollutants in concentrations deemed to be carcinogenic, mutagenic, harmful to humans and animals, etc.

The Safe Drinking Water Standards must be met at the boundary of the ZOD. The owner/operator of the ZOD installation must install monitoring wells to ensure compliance.

WATER QUALITY AT THE SITE (Table 1)

Water quality of the Floridan aquifer beneath the REF site reflects the soluble nature of the limestone substrate. The pH is at or near neutral and remains stable. The water is high in dissolved solids, with calcium, magnesium, sodium and chloride being the predominating ionic species. The latter two elements are present in high concentrations due to saline water encroachment. Due to past overdevelopment of wellfields in Pinellas County, salt water was able to intrude on the formally fresh aquifer. Other than the secondary standards of chloride and sodium, the Safe Drinking Water Standards in the Floridan aquifer are being met. The marked difference in water quality between the surficial aquifer and the Floridan aquifer implies that there is no "short-circuit", or direct connection of the two ground water bodies.

In general the water in the surficial aquifer is hard (greater than 150 mg/1 as $CaCO_3$), has an alkaline pH, and is moderately high in dissolved solids. Again, the primary solids involved are chloride and sodium. Background concentrations of iron, color and turbidity characteristically exceed the Safe Drinking Water Standards. Sulfate is also found in high concentrations.

The reason for the alkaline pH in these waters is somewhat of a mystery. It is speculated that the calcareous materials in the underlying strata, such as shell and chert, may be responsible; in general, however, the sediments of the surficial deposits are not reactive. Nonetheless, the mobility of metals in the surficial aquifer is greatly influenced by this pH condition.

Elemental concentrations in excess of the Safe Drinking Water Standards

TABLE 1

Floridan Aquifer Water Quality

Parameter	WATSTORE	Mean	N	Max
Total Organic Nitrogen, MG/L	00605	0.25	12	0.49
Total Ammonia Nitrogen, MG/L	00610	0.32	6	0.66
Total Nitrite Nitrogen, MG/L	00615	0.01	6	0.02
Total Nitrate Nitrogen, MG/L	00620	0.75	6	2.9
Total Phosphorus, MG/L	00665	0.31	14	0.83
BOD, MG/L	00310	2.85	13	7.3
COD, MG/L	00340	25	4	30
Color, PtCU	00080	11	4	20
Specific Conductance, Micromhos	00095	1342	12	1600
Dissolved Chloride, MG/L	00940	198	11	230
pH	00400	7.5	13	8.0
Dissolved Calcium, MG/L	00915	126	12	150
Dissolved Magnesium, MG/L	00925	30	12	34
Dissolved Sodium, MG/L	00930	107	12	120
Dissolved Potassium, MG/L	00935	15	11	16
Hardness, Ca MG/L	00900	436	12	390
Total Arsenic, UG/L	01002	2	12	6
Total Cadmium, UG/L	01027	0.67	12	3
Total Chromium, UG/L	01034	11	11	20
Total Copper, UG/L	01042	25	12	45
Total Iron, UG/L	01045	2562	11	4400
Total Lead, UG/L	01051	50	12	160
Total Mercury, UG/L	71900	0.19	10	0.6
Total Zinc, UG/L	01092	55	12	150
Coliform MPN	31505	287	9	2400
Fecal Coliform MPN	31615	112	11	1100

TABLE 1

Surficial Aquifer Water Quality

Parameter	WATSTORE	Mean	N	Max
Total Organic Nitrogen, MG/L	00605	0.77	12	0.90
Total Ammonia Nitrogen, MG/L	00610	0.34	8	0.43
Total Nitrite Nitrogen, MG/L	00615	0.004	8	0.01
Total Nitrate Nitrogen, MG/L	00620	0.04	8	0.32
Total Phosphorus, MG/L	00665	0.15	14	0.32
BOD, MG/L	00335	1.12	13	3.9
COD, MG/L	00340	74	12	110
Specific Conductance, Micromhos	00095	1402	15	1700
Dissolved Chloride, MG/L	00940	214	13	280
pH	00400	7.7	13	8.2
Dissolved Calcium, MG/L	00915	134	11	160
Dissolved Magnesium, MG/L	00925	21	11	26
Dissolved Sodium, MG/L	00930	146	11	160
Dissolved Potassium, MG/L	00935	1.7	11	1.9
Hardness, Ca MG/L	00900	420	11	510
Total Arsenic, UG/L	01002	8	12	11
Total Cadmium, UG/L	01027	0.45	11	2
Total Copper, UG/L	01042	12	12	41
Total Chromium, UG/L	01034	11.7	12	20
Total Iron, UG/L	01045	6055	11	7800
Total Lead, UG/L	01051	20	12	43
Total Mercury, UG/L	71900	0.19	12	0.9
Total Zinc, UG/L	01092	51	12	150
Coliforn MPN	31505	184	14	2300
Fecal Coliforn MPN	31615	24	14	300
Color, PtCU	00080	50	12	200
Turbidity, NTU	00070	60	10	100

TABLE 1

Surface Water Quality

Parameter	WATSTORE	Mean	N	Max
Total Organic Nitrogen, MG/L	00605	0.99	10	1.3
Total Ammonia Nitrogen, MG/L	00610	0.08	8	0.13
Total Nitrite Nitrogen, MG/L	00615	0.0	8	0.0
Total Nitrate Nitrogen, MG/L	00620	0.0	7	0.0
Total Phosphorus, MG/L	00665	0.06	10	0.10
BOD, MG/L	00310	1.59	10	2.1
COD, MG/L	00340	52	10	73
Turbidity, JTU	00070	9.5	2	15
Color, PtCU	00080	53	3	65
Specific Conductance, Micromhos	00095	504	10	740
Dissolved Chloride, MG/L	00940	43	10	80
pH	00400	7.4	12	8.6
Dissolved Calcium, MG/L	00915	72	8	89
Dissolved Magnesium, MG/L	00925	7.1	8	11
Dissolved Potassium, MG/L	00935	3.5	8	6.0
Dissolved Sodium, MG/L	00930	29	8	54
Hardness, Ca MG/L	00900	210	8	270
Total Arsenic, UG/L	01002	1.8	10	4
Total Cadmium, UG/L	01027	0.2	10	1
Total Chromium, UG/L	01034	15	10	30
Total Copper, UG/L	01042	5.7	10	27
Total Iron, UG/L	01045	251	10	470
Total Lead, UG/L	01051	4.2	10	12
Total Mercury, UG/L	71900	0.28	10	0.8
Total Zinc, UG/L	01092	20	10	30
Confirmed Coliform MPN	31505	16,114	9	110,000
Fecal Coliform MPN	31615	1,074	7	4,600

have been observed at various locations in the surficial aquifer. The type of contamination is attributable to the nature of the overlying land use. For example:

1. Ground water adjacent to old landfills may contain high concentrations of cadmium, chromium, and lead. Ammonia nitrogen is above background as are values for chlorides and specific conductance. These factors suggest the presence of landfill leachate; the pH, however, remains slightly alkaline.

2. Ground water beneath the sludge disposal site contains very high concentrations of nitrate nitrogen. Concentrations of metals and dissolved solids are near background.

Surface water quality is very similar to that observed in the surficial aquifer with the exception of suspended solids. During storm periods, the suspended solids concentration can exceed 50 mg/l. As is observed for the surficial aquifer, concentrations of cadmium, chromium and lead may be in excess of the Safe Drinking Water Standards; while sodium may be less, chloride and sulfate concentrations are also similar. The pH of surface water is closer to neutral than is the shallow ground water, but is still slightly alkaline.

STRATEGY FOR COMPLIANCE

The strategy for compliance with these regulations is composed of conceptual and technical solutions. The first step is to organize the pollution sources under a single site permit. Step two is to construct a slurry wall around the entire site which is keyed into the aquiclude. Step three is to install a drainage system within the wall which conveys water to the plant's cooling system. The final step is to upgrade the sites's aeration basin to accept collected leachate.

The compliance program is designed to meet the following criteria:

1. Zero discharge of surficial ground water in the ZOD to both adjacent shallow ground waters and the Floridan aquifer.

2. Minimize discharges of stormwater from the site to surrounding "waters of the State."

3. In the compliance design, maintain as much land as possible for future landfilling purposes.

4. Provide a method for treatment of landfill leachate and, possibly, stormwater collected within the ZOD.

5. Site surface waters collected for cooling purposes at the plant must meet the quantity and quality criteria stipulated in the operating agreements between Pinellas County (REF owner) and the plant operator.

6. A monitoring system must be implemented to evaluate the effectiveness of the compliance program. 6

Step 1 - There are two strategies by which the compliance program can be permitted; District Department of Environmental Regulation (DER) permits, or certification under the Florida Power Plant Siting Act.

District DER jurisdictional authority for the compliance program requires permits for stormwater, landfill operation, a ground water monitoring plan, landfill closure, monitoring well construction, and dredge and fill operations. With the exception of the monitoring well permits, each one requires renewal within a maximum period of five years.

The REF site was originally permitted under Florida's Power Plant Siting Act (PPSA), which is administered by the DER at the State capitol. In addition to certifying the electrical generation characteristics of the REF, the PPSA required a detailed environmental impact assessment. Upon receiving certification, the REF was permitted as an electrical generation facility, a solid waste incinerator, a solid waste and residue landfill, a stormwater discharger, and an air pollution source. However, the original REF site did not include certain lands described in preceding sections, such as the sludge land application site.

The primary advantage to having PPSA certification is that all potential pollution sources, be they solid, liquid, or gas related, are permitted by a single process. Another advantage is that the permit is valid for the life of the REF, although the DER continuously reviews its operating status. Therefor, an expansion of the original PPSA certification is the selected permitting strategy; the expanded certified site will include the sludge application site and the fluidized bed incinerator.

Step 2 - It is desired to contain all ground water within the ZOD; a detailed hydrogeologic survey conducted at the site revealed that the underlying confining layer is areally continuous and is of low permeability. To minimize horizontal ground water flow outward from the ZOD, a bentonite-soil slurry wall is being designed. The wall will be constructed around the entire REF site utilizing the trench-backfill method. This technique features a backhoe which excavates a trench along the wall right-of-way and places the excavated material alongside the trench. As the trench is being dug, it is filled with a slurry of pure bentonite and water. This serves two purposes. First, it stabilizes the trench wall and prevents sloughing; second, it allows the bentonite to form a low permeability "cake" on the trench sidewalls. A bulldozer then mixes the soil piled on the side of the trench with pure bentonite. Based on laboratory tests, a 2% bentonite concentration will provide the desired permeability range of 10^{-6} to 10^{-7} cm/sec. This mixture is then backfilled into the trench to final elevation.

In total, the wall will be approximately 3 feet thick. The low permeability of the wall will be due to the pure bentonite cake on the trench sidewall and the three foot thick 2% bentonite-soil wall. The wall will be keyed into the underlying confining layer to a minimum depth of 3 feet. Using an average depth to this layer of 45 feet, and a total wall length of 27,000 linear feet, over 1.2 million square feet of wall surface area will be enplaced, making this project one of the largest ever constructed.

A key element of wall construction is that the soil-slurry barrier must be properly tied into the underlying confining layer. To ensure this, an independent consultant will continuously monitor the trench cuttings as they are brought to the surface. In addition, borings are being taken on random centers along the wall route.

Constant head permeability tests using site leachate were conducted on 2% bentonite-soil solutions. The tests were designed to assess any changes in wall permeability to "chemical attack" (e.g. ionic transfer, etc.). The site leachate was subsequently analyzed for all priority pollutants; these tests were conducted to characterize the liquid, especially in regards to reactive and volatile species.

The tests have shown that no significant changes in wall permeability occur after continuous exposure to the leachate over a minimum period of 30 days. Furthermore, no chemical constituents were detected in the leachate which could compromise the wall's physical integrity or which are more penetrating than the liquid leachate (e.g. volatile organics).

Step 3 - Once the slurry wall is in place, the entire ZOD will essentially become a bathtub. Given that most of the annual rainfall at the site (54") falls in a 6 month period, the water level inside will rise to abnormally high levels. This is not acceptable due to the following reasons:

1. Stormwater (of questionable quality) will be discharged to "waters of the State."

2. The water table elevation in the surficial aquifer within the wall will be higher than that on the outside. This will form an outward gradient which promotes migration of ground water out of the ZOD.

3. The raising of the water table will increase the driving head between the surficial aquifer and the underlying Floridan aquifer. This promotes the downward movement of ground water through the confining layer.

4. The site will become flooded which interferes with day-to-day operations and promotes mosquito breeding.

It is desired to maintain, at a minimum, equivalent water table elevations on both sides of the slurry wall. In this manner, the wall is virtually impermeable. To do this, a minimum of 350 gpm of water must be continuously discharged from the site.

The only feasible method of discharge is to employ site waters for use in the REF's cooling towers. These mechanical updraft units currently utilize tertiary wastewater effluent and are capable of discharging 750 gpm to the atmosphere.

As previously discussed, surface water and ground water in the shallow aquifer are virtually indistinguishable. Surface water at the REF site is drained to a single holding pond via lateral ditches and canals. By lowering the level of the holding pond, the water levels in the ditches and canals are subsequently lowered. This induces ground water flow into the system. The lower the water in the surface drainage system, the faster the inflow of ground water. This is analagous to the development of a wellfield. What is sought is the establishment of an equilibrium, with multiple cones of depression around ditches, canals, and the holding pond.

The quality of water in the holding pond was evaluated over a variety of climatological conditions. Of concern are the typical parameters for industrial processes such as alkalinity, chlorides, sulfate, hardness, pH and orthophosphate. In addition, since a portion of the flow through the cooling towers is concentrated and discharged to the sanitary sewer, there is concern that heavy metal constituents in the flow may violate the County's sewer ordinance.

Based on the sampling regimen results, it is feasible to employ surface waters for cooling purposes. While the chloride and sulfate levels were of some concern, the overall quality was equal to or exceeded that for the current supply of tertiary wastewater.

The stormwater delivery system to the cooling towers is composed of intake structures at the site holding pond, pumps, piping and flow regulators, and a continuous backwash filter. Water will be conveyed directly to the cooling tower recirculation basin after disinfection. Filter backwash water will be discharged to the holding pond.

Step 4 - It is anticipated that the ground water quality within the slurry wall will eventually deteriorate. The contaminants involved will be primarily mineral (e.g. chlorides, sulfate), although heavy metals, such as cadmium and lead, may become a problem.

Recently, tests results were published which conclude that the aeration of raw leachate from the site in an existing REF basin promoted a 45% drop in specific conductance of the liquid. This reduction was attributed to the oxidation of soluble iron to more insoluble forms. It is further speculated that certain minerals, such as chloride and sulfate, may exhibit similar behavior during aeration. Based on these results, bench scale tests are now being designed to evaluate the following:

1. The behavior of dissolved solids in response to various levels of aeration (time duration, pH level, bubble size).
2. The settling properties of solids flocculated during aeration, including the effects of settling aids.
3. The behavior of heavy metals during aeration.
4. The accumulation rate of settled solids.
5. The toxicity characteristic of settled solids.
6. The behavior of nutrients during aeration emphasizing

ammonification and nitrification.

CONCLUSIONS

A wide variety of waste materials is being concentrated at the Pinellas County Refuse to Energy site. Both ground water and surface water quality has shown some deterioration which is characteristic of the adjacent land use types. To comply with Florida's stringent ground water regulations, a bentonite-soil slurry wall is being designed for construction around the entire site; the wall will be keyed into a continuous underlying confining layer of low permeability. To control flooding inside the wall and thereby minimize outward gradients to surrounding ground water, site waters will be continuously pumped to the plant's cooling towers. Recognizing that the water quality inside the wall will eventually deteriorate, the use of aeration and combined flocculation/sedimentation as a treatment method is being investigated.

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